# PROCEEDINGS

OF THE

# AMERICAN SOCIETY

OF

# CIVIL ENGINEERS

VOL. XLIII—No. 5

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May, 1917

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PROCEEDINGS

**This Society is not responsible for any statement made or opinion expressed in its publications.**

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## MINUTES OF MEETINGS

### OF THE SOCIETY

**April 18th, 1917.**—The meeting was called to order at 8.30 P. M.; Director Edwin Duryea in the chair; Chas. Warren Hunt, Secretary; and present, also, 134 members and 9 guests.

A paper by L. R. Jorgensen, M. Am. Soc. C. E., entitled "Multiple-Arch Dams on Rush Creek, California", was presented by William P. Creager, M. Am. Soc. C. E., and illustrated with lantern slides. The paper was discussed orally by Messrs. F. O. Blackwell, A. D. Flinn, F. W. Scheidenhelm, Edward Wegmann, W. J. Douglas, and Edwin Duryea.

A second paper, entitled "Cement Joints for Cast-Iron Water Mains," by Clark H. Shaw, Assoc. M. Am. Soc. C. E., was presented by the Secretary, who also read a communication on the subject from Harry Y. Carson, Jun. Am. Soc. C. E. Written discussions from

Messrs. F. M. Randlett and Walter Pearl were read by title, and the paper was discussed orally by Messrs. H. G. Moulton and T. Kennard Thomson.

On motion, duly seconded, a vote of thanks was given to Mr. Creager for his kindness in presenting Mr. Jorgensen's paper.

The Secretary called attention to certain resolutions,* relating to universal military training and service, adopted unanimously by the Board of Direction at its quarterly meeting of April 18th, 1917.

The Secretary announced the election of the following candidates on April 17th, 1917:

As Members

David Arthur Allee, Schnectady, N. Y.
John Walter Frink Bennett, New York City
James Madison Garrett, Montgomery, Ala.
Norman Rothwell Gibson, Niagara Falls, Ont., Canada
Glenwood Lyle McLane, Hutchinson, Kans.
John Matthew Rayburn, Pittsburgh, Pa.
Henry Aldis Varney, Brookline, Mass.

As Associate Members

Justin Frederic Barber, Groveland, Cal.
Roy Ross Clark, Portland, Ore.
Howard Wilbur Congdon, Providence, R. I.
John Hoffman Dunlap, Iowa City, Iowa
Ramón Salas Edwards, Santiago, Chile
Howard Richards Farnsworth, Neligh, Nebr.
Robert Marshall Fraser, Rome, N. Y.
Albert Webster Galbreath, St. Louis, Mo.
Otis Gibson, San Francisco, Cal.
Frank Peat Goeder, Fort Collins, Colo.
Robert Rutledge Gould, Brooklyn, N. Y.
George Appleton Griffin, New York City
Walter Alexander Heimbuecher, University City, Mo.
Edgar Malin Hoopes, Jr., Wilmington, Del.
Alfred Albert Hormel, New Haven, Conn.
Robert Bruce Houston, Kansas City, Mo.
Chester Robert Hunt, Oakland, Cal.
John Preston Irwin, Lexington, Va.
George Holt Jamison, Chicago, Ill.
Halbert Theodore Johnson, Nixon, Nev.
James Kearney, Albany, N. Y.
Joseph Meltzer, Springfield, Mass.
Alonzo Orran Peabody, Lewiston, N. Y.

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William Frederick Peters, Medina, Ohio
LeRoy Masters Pharis, Salt Lake City, Utah
Clyde Beethoven Pyle, Sciotoville, Ohio
Franklin Kearns Rader, Lewisburg, W. Va.
Robert Bright Ray, Bakersfield, Cal.
Thomas William Secrest, Anchorage, Alaska
Charles Monroe Slaymaker, East St. Louis, Ill.
Maxwell Waide Smith, Granite City, Ill.
Seymour Standish, Chicago, Ill.
Amzi Raymond Swem, Kenwood Park, Iowa
Harold Alexander Taylor, Pittsburgh, Pa.
Bernhardt E. Torpen, Aberdeen, Wash.
Lyman Wise Ward, Goldendale, Wash.
Frank Harris Wells, Detroit, Mich.
Clifford Asher Woodworth, Ida Grove, Iowa
Claude Richard Wright, Portland, Ore.
Robert Elgene Yolton, Birmingham, Ala.

As Juniors

Herbert Ashford Robertson Austin, Honolulu, Hawaii
Ralph Douglass Barnes, Batavia, N. Y.
Edwin Prescott Bly, San Francisco, Cal.
Horatio Whittemore Brown, Cambridge, Mass.
Donald Power Denham, Kansas City, Kans.
Walter Stratton Easterly, Brooklyn, N. Y.
Howard Langdon King, New York City
William Arnold Knost, Bocas del Toro, Panama
Frederick George Merckel, New York City
Charles William Pierce, Los Angeles, Cal.
James Wilson Rader, Lewisburg, W. Va.
Clifford Hoey Stem, New Orleans, La.

The Secretary announced the transfer of the following candidates on April 17th, 1917:

From Associate Member to Member

Clarence Neelly Black, Houston, Tex.
Ernest Bateman Black, Kansas City, Mo.
John George Lawrence Cunningham, Spokane, Wash.
Carson Geyer French, Cleveland, Ohio
William Garrett Grove, New York City
Herbert Miller Hale, New York City
Clifford Milburn Holland, Brooklyn, N. Y.
Alfred Majendie Lund, Little Rock, Ark.
William Hopf Popert, San Francisco, Cal.

Frank Alfred Randall, Chicago, Ill.
Horace Prettyman Warren, Seattle, Wash.
Andrew Weiss, Mitchell, Nebr.
Wesley Akers Wynn, Harrisburg, Pa.
Charles Asa Dilts Young, Seattle, Wash.

From Junior to Associate Member

John Edward Anderson, France
Ralph Ernest Beck, Brooklyn, N. Y.
Paul Caldwell Campbell, Kansas City, Mo.
Jacob Xenab Cohen, Syracuse, N. Y.
William Owen Cotton, Idaho Falls, Idaho
Arthur Klock Hinds, New York City
Guy G. Mills, Atlanta, Ga.
Walter Scott Obermeyer, Pittsburgh, Pa.
Charles Bach Seib, Kingston, N. Y.
Elroy George Smith, Augusta, Ga.
Roy Elmer Smith, Seattle, Wash.
Robert L'Hommedieu Tate, Buffalo, N. Y.

The Secretary announced the following deaths:

Otis Francis Clapp (*Director*), of Providence, R. I., elected Member, March 2d, 1898; died March 3d, 1917.

Joseph Phineas Davis, of Yonkers, N. Y., elected Member, January 29th, 1868; died March 31st, 1917.

Edmund Hazen Drury, of Ottawa, Ont., Canada, elected Member, October 4th, 1905; died January 31st, 1917.

William Henry Hunter, of Manchester, England, elected Member, February 7th, 1906; died February 27th, 1917.

William Herbert Hyde, of Washington, D. C., elected Junior, April 30th, 1901; Associate Member, June 4th, 1902; died April 15th, 1917.

Adjourned.

**May 2d, 1917.**—The meeting was called to order at 8.30 P. M.; Director Alfred D. Flinn in the chair; Chas. Warren Hunt, Secretary; and present, also, 92 members and 9 guests.

The minutes of the meetings of March 14th, March 21st, and April 4th, 1917, were approved as printed in *Proceedings* for April, 1917.

A paper by F. C. Carstarphen, Assoc. M. Am. Soc. C. E., entitled "An Aerial Tramway for the Saline Valley Salt Company, Inyo County, California", was presented by the author and illustrated with lantern slides. The subject was discussed by Messrs. Richard Lamb, H. F. Scholtz, and the author.

The Secretary announced the following deaths:

ARTHUR BEARDSLEY, of Swarthmore, Pa., elected Associate, September 1st, 1875; Member, September 2d, 1891; date of death unknown.

VAN BRUNT BERGEN, of Brooklyn, N. Y., elected Member, June 17th, 1868; died April 27th, 1917.

ERLE LEROY VEUVE, of Los Angeles, Cal., elected Junior, September 3d, 1901; Member, February 2d, 1909; died March 25th, 1917.

EDWARD THOMAS WRIGHT, of Los Angeles, Cal., elected Member, February 3d, 1886; died March 29th, 1917.

HAROLD DAVIS, of Washington, D. C., elected Associate Member, October 2d, 1901; died March 25th, 1917.

Adjourned.

## OF THE BOARD OF DIRECTION

(Abstract)

**April 17th, 1917.**—The Board met at 10.05 A. M., President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Crocker, Darling, Davies, Davis, Duryea, Flinn, Harwood, Hawley, Herschel, Humphreys, Khuen, McDonald, Marx, Noble, Ockerson, Ricketts, Rights, and Webster.

Report of Committee on Revision of the Constitution made first order of business.

Mr. Endicott was invited to attend the Board meeting and present the Report of the Committee of which he is Chairman.

Mr. Endicott presented the revised Constitution as adopted by the Committee, also a Statement signed by himself, Messrs. Ockerson and McDonald.

The revised Constitution as presented by the Committee was considered article by article and amended.

Vice-President Kittredge and Treasurer Tillson came in while this action was going on.

On motion, duly seconded, the revised Constitution, as reported by the Committee, and amended by the Board, was ordered sent out to the membership under Article IX of the Constitution.*

On motion, duly seconded, the Committee on Revision of the Constitution was discharged.

Adjourned to meet at 2.15 P. M.

2.15 P. M. the Board reconvened.

On motion of Mr. McDonald, seconded by Mr. Ockerson, the entire correspondence with relation to the Constitution was ordered spread upon the Minutes.

The Secretary reported the death of Director Otis F. Clapp, on March 3d, 1917.

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* Page 315.

On motion, duly seconded, Frederic H. Fay was nominated and unanimously elected a Director of the Society to fill the vacancy caused by the death of Director Otis F. Clapp.

Director Harwood presented his resignation as Chairman of the Library Committee, and Director Flinn was appointed as Chairman of the Library Committee.

The Secretary reported that at the request of the Western Society of Engineers President Pegram had appointed Messrs. Onward Bates and William H. Finley as representatives of this Society on a Board to make the "Washington Award."

Mr. Hunt of the Committee on the Alfred Noble Memorial reported progress.

The question of the proposed enlargement of the functions of the Joint Committee on National Engineer Reserve was referred to the President with power.

A report was presented from a Committee on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to Other Engineering Organizations, and Engineers, and to the Public, which on motion, duly seconded, was received, its recommendations adopted, and ordered printed in *Proceedings* and discussion invited.*

The Secretary reported that since the last meeting of the Board all the books in the Library, except duplicates, had been moved to the U. E. S. Building and turned over to the Joint Library; that there remain on our shelves about 22 000 duplicates; that numerous requests have been received for donation of these books to various Libraries. The whole matter of the disposition of the duplicates in the Library was referred to the Executive Committee with power.

The following was adopted:

*"Resolved:* That ownership of the Library of the American Society of Civil Engineers now in the Library of the United Engineering Society be transferred to that Society whenever the necessary legal steps for such transfer are made."

A proposal for the formation of an Engineering Council forwarded by a General Joint Committee appointed for the purpose, in the form of suggested amendments to the By-laws of the U. E. S., was presented.

The suggested amendments were approved as presented.†

The following resolutions were adopted:

*"Resolved:* That it is the sense of the Board of Direction of the American Society of Civil Engineers that the representation of the U. E. S. on the Engineering Council should be four instead of three, and that the words 'Governing Bodies of each of the Founder Societies, and to the' be added in the last line of the proposed amendment to the By-law, so that the last sentence shall read: 'The Council shall keep

* See p. 327.
† See p. 342.

a record of its proceedings and transmit, after each meeting, a copy of the same to the Governing Bodies of each of the Founder Societies, and to the Board of Trustees of the United Engineering Society'; provided the United Engineering Society may find it possible to make these changes in the proposed suggestions."

Correspondence relating to the propriety of the acceptance by an Engineer of a contingent fee was presented.*

The Secretary was instructed to publish this correspondence in *Proceedings.*

A report was received from Messrs. F. G. Jonah and C. L. Strobel, Delegates appointed to attend the Conference on Engineering Co-operation, Chicago, March 29th and 30th, 1917.

A revision of the Constitution of the Baltimore Association of Members was presented and approved.

Letter from the San Diego Association of Members relating to a proposed Bill relative to the licensing of architects in California, together with a copy of a resolution unanimously adopted by the San Diego Association condemning it, was presented, and the publication of this matter in *Proceedings* was authorized.†

The proposed Constitution of the Duluth Association of Members was presented and approved by the Board.

Letter from the Secretary of the Military Engineering Committee of New York requesting co-operation of this Society, and suggesting the appointment of one or more representatives to confer with this Committee was presented. The Secretary was instructed to reply that as this Society is a National Society it is not within its province to take the action indicated, which relates to a local matter.

Letter from the Secretary of the Department of Commerce urging co-operation with the Department of Agriculture in its campaign to increase the crops of the country was presented, and was ordered published in the May Number of *Proceedings.*‡

The following resolution was adopted:

"*Resolved:* That no officer of the Society shall officially recommend any one for any office or position."

The resignation of Frank C. Osborn as a member of the Special Committee on Steel Columns and Struts was received and accepted.

The resignation of J. B. Berry as a member of the Special Committee on Stresses in Railroad Track was presented and accepted.

The Secretary reported that, in accordance with the authority of the Board, Chairman Bates had appointed Hugh L. Cooper a member of the Alfred Noble Memorial Committee.

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* See p. 332.
† See p. 334.
‡ See p. 333.

The following resolution was adopted:

"*Resolved:* That the Executive Committee be given power to pass upon all routine matters, and those requiring prompt attention and which do not affect the general policy of the Society, and that the Secretary forward an abstract of such action by the Executive Committee to each member of the Board."

Action was taken in regard to members in arrears for dues.

The resignations of 1 Member, 3 Associate Members and 4 Juniors, were accepted.

Ballots for membership were canvassed, resulting in the election of 7 Members, 40 Associate Members, and 12 Juniors, and the transfer of 12 Juniors to the grade of Associate Member.

Fourteen Associate Members were transferred to the grade of Member.

Adjourned 8 P. M.

**April 18th, 1917.**—The Board met at 10.20 A. M., President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Crocker, Davies, Duryea, Flinn, Hawley, Herschel, Khuen, Kittredge, McDonald, Noble, and Rights.

A Report from Committee appointed to report on the Establishment by the Society of an Employment Bureau, Messrs. F. H. Newell, H. S. Crocker, and E. N. Layfield, was received, and the Report was accepted, the Committee discharged, and the Report ordered printed in *Proceedings*.*

On motion, duly seconded, the representatives of the Society on the U. E. S. Board were instructed to say that the Board of Direction is in favor of a central committee to carry out the matter suggested in this Report.

A resolution was adopted in regard to universal military training.†

Adjourned.

The Board reconvened at 5.50 P. M., President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Duryea, Flinn, Khuen, Kittredge, McDonald, Noble, and Tillson.

This meeting was held subsequent to the meeting of the Membership Committee which had continued all day since the adjournment of the previous Board meeting.

A report from the Membership Committee was received and acted upon.

The date of the next Board meeting was fixed as Monday, June 11th, at 10 A. M.

Adjourned 6.05 P. M.

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* See p. 330.
† See p. 331.

# SOCIETY ITEMS OF INTEREST

## Proposed Revision of the Constitution

On April 17th, 1917, a Committee of the Board appointed to prepare a revision of the Constitution reported to the Board of Direction. The Board considered the Report, and ordered that the revised Constitution, as prepared by the Committee, and amended by the Board, be sent out to the membership under Article IX of the Constitution.

The revision of the Constitution, printed below in accordance with this resolution, will, under the provision of Article IX of the Constitution, be brought before the Annual Meeting of the Society to be held January 16th, 1918.

## REVISED CONSTITUTION

### ARTICLE I.—Name, Location and Object.

1.—The name of this Association shall be the American Society of Civil Engineers.

2.—The office of the Society shall be in the City of New York.

3.—Its objects shall be the advancement of engineering knowledge and practice, the maintenance of high professional standards among engineers, and active participation in affairs which involve engineering interests.

4.—In furtherance of these purposes meetings for the presentation and discussion of appropriate papers and topics, and for social and professional intercourse, shall be held; adequate facilities for the transaction of its business and for meetings of its members shall be provided; publications may be issued, and libraries maintained.

5.—Subsidiary Associations, limited to the membership in all grades may be organized to promote social and professional intercourse among the membership, and to stimulate interest and encourage active participation in the objects of the Society. Such Associations may be formed with any geographical limitation desired by any group of members, when such formation and limitation are approved and authorized by the Board of Direction. Such Associations shall be governed by Constitutions approved by said Board.

### ARTICLE II.—Membership.

1.—The Corporate Members shall be designated as Members and Associate Members. There may also be connected with the Society, Honorary Members, Affiliates, and Juniors.

2.—A Member shall be an engineer in any branch of the profession, who at the time of admission shall be not less than thirty-two years of age, and shall have been in the active practice of the profession for not less than ten years; shall have had responsible charge of the construc-

**tion of engineering work of high grade for at least five years, and shall have designed work of such character.**

**3.—An Associate Member shall be an engineer in any branch of the profession, who at the time of admission shall be not less than twenty-five years of age; shall have been in the active practice of the profession for not less than six years; and shall have had responsible charge of engineering work as principal or assistant for at least one year.**

**4.—Graduation from a school of engineering of high standing shall be considered as equivalent to two years of active practice. The performance of the duties of a Professor of Engineering in such a school shall be considered as equivalent to the same time spent in actual practice, but shall not be considered as time spent in responsible charge of engineering work.**

**5.—An Affiliate shall be a person of distinction, who is qualified to co-operate with engineers in the advancement of professional knowledge and practice.**

**6.—A Junior shall be not less than twenty years of age when admitted, and shall have had active practice in some branch of engineering for at least two years, or shall have been graduated from an engineering school of high standing. Connection with the Society as Junior shall cease at the age of thirty-four.**

**7.—Affiliates and Juniors shall be entitled to all the privileges of the Society, except the right to vote and to hold office therein.**

**8.—Honorary Members shall be persons of eminence in some branch of Engineering or the sciences related thereto. There shall not be more than twenty Honorary Members at any one time.**

**9.—The status of any member of the Society in any grade, at the time of the adoption of this Constitution, shall not be affected in any way by its provisions, except that the Associates at that time shall become Affiliates.**

**ARTICLE III.—Admissions, Discipline and Expulsions.**

**1.—Admission to the Society shall be by vote of the Board of Direction, which shall have the sole power to elect persons to any grade, to transfer from one grade of membership to another, and to discipline or to expel members of any grade in accordance with the provisions of this Constitution, and such by-laws as may be prescribed by said Board. The action of the Board in all such matters shall be reported to the Society.**

**2.—All persons elected and duly qualified, whose address on the records of the Society is within fifty miles of the Post Office in the City of New York, shall be deemed Resident; and those whose address is beyond that limit shall be deemed Non-Resident.**

The classification of each person for the fiscal year, as Resident or Non-Resident, shall be determined by the Records of the Society as they may appear on January 1st of that year.

3.—A member of any grade in the Society may resign from membership by a written communication to the Board of Direction; when, if all dues have been paid, the resignation shall be accepted.

### ARTICLE IV.—Dues.

1.—The entrance fees payable on admission to the Society shall be as follows: By Members, thirty dollars; Associate Members, twenty-five dollars; Affiliates, twenty dollars; Juniors, ten dollars.

2.—The annual dues payable by Members, whether Resident or Non-Resident, shall be as follows: By Corporate Members, fifteen dollars; Affiliates, ten dollars; Juniors, ten dollars.

3.—In addition to the dues prescribed in the preceding section, persons living in District No. 1 shall pay annually as follows: Corporate Members, ten dollars; Affiliates, five dollars; Juniors, five dollars.

4.—A person transferred from any grade to a higher one shall pay the difference between the entrance fees of the two grades, and his annual dues shall be those of the higher grade.

5.—The annual dues shall be payable in advance on the first day of January of each year. Persons elected after six months of any fiscal year shall have expired, shall pay one-half of the amount of dues for that fiscal year.

6.—All future dues may be compounded by a single payment by a Corporate Member of $250; or by an Affiliate of $150. Should a compounding Affiliate be elected to Corporate Membership he shall pay the additional sum of $100.

Provided, that all compounding Corporate Members or Affiliates who may be or hereafter become Resident, shall be and remain liable for the annual payment of the difference between the annual dues of Resident and Non-Resident Corporate Members, or Affiliates; but any Corporate Member may at any time compound for the future payment of all annual dues of every nature and kind by the payment of $75 in addition to the $250 hereinbefore named, and any Affiliate may at any time compound for the future payment of all annual dues as Affiliate by the payment of $40 in addition to the $150 hereinbefore named.

Provided, that any person desiring to compound for future dues shall have paid his entrance fee, all arrears of dues, and the annual dues for the current year.

Persons compounding shall sign an agreement that they will be governed by the Constitution and Laws of the Society as they are now formed, or as they may be hereafter altered, amended or enlarged; and

**that in case of their ceasing to be connected with the Society from any cause whatever, the amount theretofore paid by them for compounding, and for entrance fees and annual dues, shall be the property of the Society.**

**All moneys thus paid in commutation of annual dues shall be invested as a permanent fund, only the interest thereon being subject to appropriation for current expenses.**

**7.—The Board of Direction may, for sufficient cause, temporarily excuse from payment of annual dues any member who from ill health, advanced age, or other good reason assigned, is unable to pay such dues; and the Board may remit the whole or part of dues in arrears.**

**8.—Every person admitted to the Society shall be liable for the payment of all dues until connection therewith shall have ceased.**

**9.—The status of any present Fellow or Subscriber shall not be changed by the provisions of this Constitution.**

**10.—Corporate Members and Affiliates who have paid dues as such for thirty-five years shall be exempt from further dues.**

## ARTICLE V.—OFFICERS.

**1.—The officers of the Society shall be a President, four Vice-Presidents, eighteen Directors, a Secretary, and a Treasurer. These officers, with the exception of the Secretary, together with the latest five Past-Presidents who continue to be members, shall constitute the Board of Direction in which the government of the Society shall be vested, and shall be the Trustees as provided for by the laws under which the Society is organized.**

**2.—The terms of office of the President, Secretary, and Treasurer shall be one year; of the Vice-Presidents, two years; and of the Directors, three years.**

**The term of each officer shall begin at the close of the Annual Meeting at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.**

**3.—A vacancy in the office of President shall be filled by the senior Vice-President, who shall become the President for the unexpired term, and perform all the duties pertaining to the office, and to the office of Past-President for the succeeding five years.**

**A vacancy in the office of Vice-President shall be filled by the senior Director (eligible under Article V, Section 5), who shall become the Vice-President for the unexpired term and perform all the duties pertaining to the office. Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to Corporate Membership; and when the latter dates are identical, the selection shall be made by lot.**

**In case of the disability or neglect in the performance of duty, of any officer of this Society, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided in this Section.**

**4.—The President shall be ineligible for re-election. The Vice-Presidents and Directors shall not be eligible for re-election to the same office until at least one full term shall have elapsed after the end of their respective terms.**

**5.—Only one Vice-President and six Directors shall be Resident** Corporate Members.

6.—The Secretary and the Treasurer shall be Resident Corporate members during their terms of office.

ARTICLE VI.—MANAGEMENT.

**1.—The President shall have general supervision of the affairs of the Society. He shall preside at meetings of the Society and of the Board of Direction at which he may be present, and shall be, *ex officio*, a member of all committees except the Nominating Committee. He shall deliver an address at the Annual Convention.**

**The Vice-Presidents, in order of seniority, shall preside at meetings in the absence of the President.**

**2.—The Board of Direction shall manage the affairs of the Society in conformity to the laws under which it is organized and the provisions of the Constitution, and shall adopt by-laws which it may amend from time to time and shall inform the membership of such by-laws and any amendments made thereto. It shall direct the investment and care of the funds of the Society; make appropriations for specific purposes; act upon applications for membership as heretofore provided; take measures to advance the interests of the Society; appoint all its employees; and generally direct its business. It shall make an annual report to the Annual Meeting.**

**3.—The Treasurer shall receive all moneys and deposit the same in the name of the Society. He shall invest all funds not needed for current disbursements, as shall be ordered by the Board of Direction. He shall pay all bills, when certified and audited, as provided by this Constitution and by rules to be prescribed by the Board of Direction. He shall make an annual report.**

4.—The Secretary shall be elected by ballot of the Board of Direction, and shall hold the office until a successor is elected, provided that a majority of the whole Board of Direction shall be required to elect the Secretary.

**Under the direction of the President and the Board of Direction, the Secretary shall be the executive officer of the Society, in charge of the correspondence, books of accounts and records, the Society quarters and contents thereof, and the supervision of all employees; he shall certify to the accuracy of all bills or vouchers, shall countersign all checks, collect and transfer to the Treasurer all moneys due the Society. He shall present annually a balance sheet of his books as of the 31st of December and shall, from time to time, furnish such financial or other statements as may be required by the Board of Direction. He will be expected to attend all meetings of the Society and of the Board of Direction, prepare the business therefor, and duly record the business thereof.**

**5.—The Secretary and Treasurer shall be paid salaries to be determined by the Board of Direction.**

**6.—The Board of Direction shall meet within twenty days after the Annual Meeting, and shall then appoint from its members a Finance Committee of five, a Publication Committee of five, and a Committee on Special Committees. At least three members of the Finance Committee, and two members of the Publication Committee shall be resident within fifty miles of New York.**

**7.—The Finance Committee shall have supervision of the financial affairs of the Society; employ an expert accountant to audit the accounts monthly; approve all bills before payment; prepare an annual budget; and make recommendations to the Board of Direction as to the investment of moneys, and other financial matters.**

**8.—The Publication Committee shall have supervision of the publications of the Society, and of contracts and expenditures connected therewith.**

**9.—The Committee on Special Committees shall have supervision of the work of all Special Committees.**

**10.—The President, Resident Vice-President, and Treasurer, together with the Chairmen of the Standing Committees on Finance, Publications, and Special Committees, shall constitute an Executive Committee, which shall have power to act on all matters when the Board of Direction is not in session, and to call special meetings of said Board. Any action of the Executive Committee shall be communicated promptly to all members of the Board.**

**11.—Special Committees may from time to time be appointed by the Board of Direction from the membership of the Society, to report on engineering subjects.**

**12.—All Committees shall report to the Board of Direction, and shall perform their duties under its supervision.**

**ARTICLE VII.—Nomination and Election of Officers.**

**1.—The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York.**

**Members in any grade who do not reside in North America shall be attached to District No. 1, for the purpose of voting for and being represented by the officers elected from that District, but shall not be otherwise considered as Residents. They shall pay Non-Resident dues.**

**Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year.**

**2.—Seven Corporate Members, not officers of the Society, shall be elected each year to serve for the next two years on the Nominating Committee. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve districts; and these with the latest five Past-Presidents of the Society, who continue to be members, shall be a committee to nominate officers for the Society.**

**The Board of Direction shall prescribe the mode of procedure for the election of this committee, and fill any vacancies which may occur.**

**This Committee shall meet at the Annual Convention of the Society, or at a time and place to be agreed upon by a majority of its members, but said meeting shall not be later than the fifteenth day of July. At this meeting this committee shall elect from among its members a Chairman and a Secretary to serve for one year beginning on the first day of the following September. At all meetings of the committee eight members shall constitute a quorum. If at any stated or called meeting of the committee there shall not be a quorum present, then such members as are present shall call an adjourned meeting for the transaction of the committee's business. This Committee shall select nominees to fill the offices named in Article V, with the exception of the office of Secretary, so as to provide, with the officers holding over, a Vice-President and six Directors, residing in District No. 1, and twelve Directors divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. A nominee or officer shall continue to represent the district from which nominated or elected, although residence may be changed to another district. Nominations under this Section shall be designated as "Official Nominations."**

**A list of nominees selected for the offices to be filled at the next Annual Election shall be presented by this committee to the Board of Direction not later than the first day of August, and the Secretary shall**

**thereupon notify immediately the nominees of their nomination and ascertain their acceptance or declination, and shall notify at once each member of the Board of the result.**

**3.—Before the fifteenth day of October the aforesaid list of nominees shall be mailed to every Corporate Member whose address is known, provided that if any person is found to be ineligible, or should a nominee decline, the names of such nominees shall not be sent out, but the Board shall substitute another name therefor by ballot, and further provided that in the event that the Nominating Committee fails to select a nominee for any office as above stipulated, the Board shall select a nominee therefor. The Board, in like manner, shall also fill any vacancies that may occur in the list of nominees up to the time the ballots are sent out. The Secretary shall notify immediately each member of the Board of any vacancy occurring in the list of nominees.**

**4.—Additional nominations for any office may be made by declaration. In the case of an additional nomination for any General Office, such nomination must be signed by at least twenty-five Corporate Members. In the case of an additional nomination for any Office representing any District, the nomination must be signed by at least twenty-five Corporate Members residing in that District. All nominations by declaration shall be accompanied by the written acceptance of the nominee and filed with the Board of Direction before the first day of December.**

**5.—At least thirty days before the Annual Meeting there shall be mailed to every Corporate Member a list which shall include the names and residences of all persons nominated in accordance with this Article, their grades of membership, and, in the case of nominees for Directors, the number of the District in which they reside. Nominations by declaration shall be distinguished from Official Nominations by distinctive marks prescribed by the Board of Direction.**

**Accompanying such list letter-ballots shall be forwarded. The form of such letter-ballots shall be prescribed by the Board of Direction. They shall be so arranged that the entire Corporate Membership may vote for all General Officers, but that representatives of any District shall be voted for only by the Corporate Members residing in that District.**

**Voters may strike out the name of any nominee printed in the ballot for whom they do not wish to vote, and may substitute therefor, in writing or by paster, the name of any other person for whom they wish to vote.**

**6.—Ballots may be sent by mail to the Secretary, or may be presented to him at the Society Headquarters. They should be enclosed in two sealed envelopes, and the outer envelope shall be endorsed by the voter's signature.**

**The Secretary shall make a list of the voters from whom ballots are received, which list shall be open to inspection by all Corporate**

Members. A voter may withdraw his ballot, and may substitute another, at any time before the polls close.

7.—The polls shall be closed at 9 A. M. on the first day of the Annual Meeting, and the ballots shall be canvassed publicly by tellers appointed by the President.

The persons who receive the greatest number of votes for each office to be filled shall be declared elected. In case of a tie between two or more persons for the same office, the Annual Meeting shall elect the officer from among the persons so tied.

The presiding officer shall announce to the meeting the names of the officers elected.

## ARTICLE VIII.—MEETINGS.

1.—During each year at least two General Meetings of the Society shall be held, as follows:

The Annual Meeting shall be held in New York City on the third Wednesday in January. At this meeting the annual reports for the previous year shall be presented, ballots for officers canvassed and results announced, and other business of the Society transacted.

The Annual Convention shall be held at such time and place as the Board of Direction shall determine.

Other General Meetings may be held at any time or place determined upon by the Board of Direction.

At General Meetings, in addition to the transaction of business, professional papers may be presented and discussed, and opportunities for social intercourse provided.

2.—Regular meetings for the transaction of business of the Society shall be held in New York City on the first Wednesday of each month, except July and August.

3.—Special meetings for the transaction of business may be called by the Board of Direction, and shall be so called upon the written request of thirty or more Corporate Members, which request shall state the purpose of such meetings. Calls for such meetings shall be issued not less than ten days in advance of the date on which they are to be held, and shall state the purposes thereof. No other business shall be taken up at such meetings.

4.—At all meetings of the Society at which business is transacted, thirty Corporate Members shall constitute a quorum.

5.—In addition to the General and the Business meetings herein provided, meetings, for the reading and discussion of papers only, shall be held as ordered by the Board of Direction.

6.—Meetings of the Board of Direction shall be held at the time of the Annual Meeting, at which meeting fifteen members shall constitute

**a quorum, and at such other times as the Board may determine, at which meetings five members shall constitute a quorum.**

**ARTICLE IX.—AMENDMENTS.**

**1.—Proposed amendments to this Constitution must be reduced to writing, signed by thirty or more Corporate Members, and presented to the Society at a Regular Business Meeting. They shall be submitted to the Corporate Membership and acted upon as follows:**

**2.—Amendments presented not less than sixty days previous to the date of a General Meeting shall be referred to the Board of Direction for approval or revision as to form or validity only. They shall then be sent by letter to the Corporate Membership not less than twenty-five days previous to said General Meeting and may by a majority vote of the Meeting be amended in any manner pertinent to the original form. They shall then be voted upon by letter-ballot, the vote to be counted at the second Regular Business Meeting subsequent to such General Meeting.**

**3.—An affirmative vote of two-thirds of all ballots cast shall be requisite to the adoption of any amendment.**

**4.—Amendments so adopted shall take effect thirty days after their adoption, provided that the officers of the Society at the time of the adoption of any amendment shall continue in office until the expiration of the terms for which they were elected.**

**PROPOSED BY-LAWS.**

**ASSOCIATIONS OF MEMBERS.**

**1.—Any Subsidiary Association, Membership in which shall be open to, and restricted to, membership in the American Society of Civil Engineers in all its grades, may be formed under Article 1, Section 5, of the Constitution.**

**2.—Geographically, such Associations may be limited to the boundaries of a Municipality, County, State or group of States, when authorized by the Board of Direction upon receipt of the written request of a majority of the total membership residing within the territory covered. The Constitution of each such Association of Members shall be approved by the Board of Direction.**

**3.—No Association of Members shall take any action as such, which is contrary to the Constitution of the American Society of Civil Engineers. It shall not participate in partisan politics in any way.**

**4.—The Board of Direction of the Society may withdraw its approval of any such Association by an affirmative vote of two-thirds of the whole Board.**

5.—The governing body of any such Association of Members shall in no case be called a "Board of Direction".

6.—Student branches, consisting of students in engineering schools, may be established under such regulations as the Board may prescribe.

ELECTIONS TO MEMBERSHIP.

1.—Honorary Members shall be elected in the following manner. The proposal for the election of any person as an Honorary Member shall be signed by at least ten Corporate Members of the Society, and shall be accompanied by a statement of the career of the nominee.

Honorary Members shall be elected by the affirmative vote of at least four-fifths of the entire Board of Direction.

2.—All other applications for admission to the Society, or for transfer from one grade to another, shall be in such form as may be prescribed from time to time by the Board of Direction. They shall be signed by the applicant, and shall contain a promise to conform to the requirements of the Constitution of the Society if elected. Each applicant shall furnish the names of at least five Corporate Members to whom he is personally known; each of these shall be requested by the Secretary to give to the Board of Direction, on a form prescribed for the purpose, the extent of the writer's personal knowledge of the applicant, and of his professional work. If at least five of the Corporate Members named as references do not furnish the requisite endorsement, the Secretary shall call upon the applicant for additional references, and not until written communications have been received from at least five Corporate Members shall the application be considered by the Board.

Applications from engineers not resident in North America, and who may be so situated as not to be personally known to five Corporate Members, may be acted upon by the Board of Direction after it has secured evidence, sufficient in its opinion, to warrant it in so acting.

3.—The Board of Direction shall issue from time to time to the entire membership of the Society a list of all applications for admission or for transfer, containing a concise statement of the record of each applicant, with a request that members transmit to the Board any information they may have which will aid in the consideration of the applications. Not less than twenty days after the issue of such list, the Board of Direction shall consider the applications so issued, shall classify the applicant, and shall vote upon admission or transfer by letter-ballot.

4.—At least twenty-five ballots, of which at least twenty shall be in the affirmative, shall be necessary for election or for transfer. In case any candidate fails of election, or of transfer, no notice of such failure shall be entered in the minutes, but the applicant shall be notified.

**A rejected applicant may file another application at any time after the expiration of one year from the date of rejection.**

**5.—All persons elected shall be notified promptly and shall subscribe to the Constitution of the Society upon a form prescribed by the Board of Direction and shall pay the entrance fee and current dues prescribed in Article IV, Sections 1, 2, 3, 4, 5, and 6. If these provisions are not complied with within six months from the date of notification of election, such election shall be void, unless the time be extended by special action of the Board of Direction.**

**Membership of any person shall date from the date of election.**

### DUES.

**1.—It shall be the duty of the Secretary to notify each member of the amount due for the ensuing year at the time of giving notice of the Annual Meeting.**

**2.—Any person whose dues are more than three months in arrears shall be notified by the Secretary. Should the dues not be paid when they become six months in arrears, the right to vote and to receive the publications of the Society shall be forfeited. Should the dues become nine months in arrears, the delinquent shall again be notified, and if such dues become one year in arrears, connection with the Society shall cease. The Board of Direction, however, may, for cause deemed by it sufficient, extend the time for payment and for the application of these penalties.**

### DISCIPLINE AND EXPULSION.

**1.—The Board of Direction may investigate any professional or other action of any person connected with the Society which may be brought to its attention, and if it deems such action desirable may appoint a special committee, either from its own members, or from the Membership of the Society, to investigate the case, and to report to the Board.**

**2.—In all cases the accused shall be informed of the charge made, and shall have opportunity for defence. The Board of Direction shall be empowered to warn the accused against repetition of the offence, to suspend from membership for any period, or to expel, but such action shall only be taken by letter-ballot of the whole Board, and at least twenty affirmative votes shall be necessary for such action.**

**3.—In any case whether the decision of the Board is favorable or unfavorable to the accused, its final action shall be made known to the entire membership of the Society.**

**Report of a Committee, appointed by the Board of Direction, on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to other Engineering Organizations, and Engineers, and to the Public.***

The Local Associations of the American Society of Civil Engineers are in many cases responding to a sentiment that there should be more co-operation with other Engineering Organizations, more influence exerted in local communities, and more intimate relations established between the Society and its non-resident members. With this end in view, the plan of district representation was approved by the conference of the Presidents of fourteen Associations in 1915, but, at the request of the Secretary, in February, for an expression of opinion, only four associations responded, and only one of these advocated the district plan. Your Committee has considered that this matter is not properly within its field of enquiry. All the Associations which did offer suggestions, however, agreed that they wanted few restrictions and freedom to deal with local affairs.

Your Committee is convinced that the future of the American Society of Civil Engineers, as well as the welfare of its members, is in many ways dependent on the success of its Local Associations, and that they should be encouraged to widen their field and strengthen themselves in all possible ways. Since the word "Association" is more properly applicable to a local organization of branches or sections of the National Societies, and because the word "Section" more nearly suggests the close relation to the Society which is sought to be established, and besides is a more convenient word, the Committee recommends that the Local Associations be known as "Sections" of the American Society of Civil Engineers.

The following Rules are substantially those which the Board of Direction has already sanctioned. The paragraphs under the heading "Policy" express the Committee's conclusions as to the conduct of the Associations which would be desirable, and which, if approved by the Board of Direction, should be sent to all the Associations as an expression of its views.

Respectfully submitted,

DANIEL BONTECOU,
HUNTER MCDONALD,
F. G. JONAH.

RULES.

*Initiation.*—A Local Section may be authorized by the Board of Direction at the written request of fifteen Corporate Members of the

* This report was presented to the Board of Direction at its meeting of April 17th, 1917, and its recommendations were adopted and ordered printed in *Proceedings*, and discussion invited.

Society, and must consist of at least twenty-five members of all grades residing within the territory covered.

*Constitution and By-Laws.*—The Constitution and By-Laws of a Local Section must be approved by the Board of Direction. The Constitution should state, in effect:

First.—The objects to be attained are the advancement of engineering knowledge and practice; the cultivation of friendly relations with all engineers; the maintenance of high professional standards; and co-operation with other engineering societies, with a view to promoting the general welfare of the American Society of Civil Engineers and the Engineering Profession.

Second.—Members of the American Society of Civil Engineers, in any grade, who qualify by paying dues and subscribing to the Constitution and By-laws of the Section, are eligible to membership, without payment of entrance fee. Any member of a Local Section who at any time ceases to be a member of the American Society of Civil Engineers, shall at the same time cease to be a member of the Section.

Third.—The Constitution may only be amended by the vote of two-thirds of the members qualified to vote, and the amendment must have first received the approval of the Board of Direction.

Fourth.—The remaining Articles of the Constitution, any By-laws, covering matters of local importance only, such as those relating to officers, meetings, dues, etc., may be largely determined by local necessity and preference; but they should include a statement that, among the means to be adopted for securing the objects of the organization are: Meetings, to promote acquaintance and good fellowship between Engineers, and, if desired, for the presentation and discussion of professional papers, either prepared by members of the section or issued by the American Society of Civil Engineers; Co-operation with other engineers, as individuals or as constituent bodies of local organizations; Participation in local and State affairs, and exercise of influence in properly solving Public Engineering Problems.

## Policy.

### *Relations of Local Sections to the American Society of Civil Engineers.*

*Reports.*—Local Sections should transmit promptly to the Secretary of the American Society of Civil Engineers an abstract of such proceedings of their meetings as may be deemed of general interest, including, when practicable, discussions of engineering subjects and reports in full. These may be printed, after editing, in the *Proceedings* of the Society.

*New Members.*—Local Sections should seek in all proper ways to increase the membership of the American Society of Civil Engineers and to advance its prestige and influence. They should investigate carefully the qualifications of any candidate for admission to the Society, about whom the Board of Direction desires information, and make an official report at the request of the Secretary of the Society.

*Restriction in Activity.*—Local Sections should refrain from all action on matters of a National character, or which might possibly affect the general interests of the Profession, without the approval of the Board of Direction, and should report to the Secretary any local matter which might affect or interest the Society or Profession at large.

*Ethics.*—Local Sections should insist on the observance of the Code of Ethics adopted by the American Society of Civil Engineers, and support the Society in matters relating to it. Charges made by any member of the Society that a member of the Section has violated this code should be promptly investigated by a committee of the Section, and its findings officially reported to the Board of Direction.

*Grievances.*—Local Sections should endeavor to protect the professional reputation of their members by investigating and publishing the facts of any case in which their integrity or competency appears to have been unfairly or maliciously misrepresented, and should report the circumstances and the action taken to the Board of Direction.

### *Relations of Local Sections to Sections of other National Societies, to Local Engineering Clubs, and to All Engineers.*

*Co-operation.*—Local Sections should adopt a broad view of their relations to the Engineering Profession, and recognize the advantage and necessity of co-operation with other engineering organizations in all ways that may strengthen the position of engineers, develop social relations among them, or tend to establish correct principles and practice.

*Local Societies.*—Local Sections should affiliate with existing Engineering Societies, and, where none exist, and the conditions are favorable, they should encourage their formation. All engineers should be considered as eligible for membership in such Societies, but only those whose attainments correspond at least to the requirements for Associate Membership in the American Society of Civil Engineers should have a vote. It is desirable that not more than 20% of the membership of these Societies should be admitted as associates, who, although not engineers, have interests in common with them. Members of a Section of the American Society of Civil Engineers should vote in such Societies only as individuals, and not as a body.

*Engineers.*—The Local Sections should endeavor to promote friendship among engineers, regardless of their connection with any Society, and to extend to them professional support. They should oppose any tendency among engineers to pay inadequate compensation to engineers whom they employ, on account of the experience to be gained by the employment, and should discourage in all proper ways persons who assume the functions of engineers without possessing their recognized qualifications.

*Relations of Local Sections to the Public.*

*Education.*—Local Sections should accept the responsibility of educating the public to a proper appreciation of the services of engineers in all business which involves their experience and knowledge.

*Public Affairs.*—Local Sections should volunteer judicious and carefully considered advice on public matters involving engineering questions, but this advice must be above any suspicion of connection with personal advantage. It is undesirable that there should be any encroachment on the practice of individual engineers or participation in personal or partisan politics, and it is of the highest importance to the Profession that engineers should realize and accept their duties as citizens of the community in which they live.

*State Affairs.*—Local Sections need not be confined to State lines, but they should endeavor to arrange some organization of the engineers of the States within their territory, for the purpose of exerting influence in the legislation of the State and the administration of its affairs wherever engineering principles or practices are involved.

## Report of Committee on Employment Bureau

"April 13, 1917.

"To the Board of Direction,
American Society of Civil Engineers.

"Gentlemen:—Your Committee, appointed in July, 1916, to consider and report upon the question of the desirability of the establishment of some form of an employment bureau, in connection with the activities of the Society, explains its delay in reporting as due to the difficulty in securing a meeting at which all the members could be present, but, having at last been able to hold such a meeting, desires to report as follows:

"First.—Replies to inquiries addressed to many engineering societies and to individual engineers, especially to our younger members, also the discussion that took place at the Annual Meeting last January, and the recent formation of various organizations of engineers with the avowed object of securing employment, all indicate a wide-spread demand for action by the societies in this matter. It is further believed that there is in existence a general feeling that our Society

should, in the future, give systematic study to the practical matter of employment.

"Second.—Should the American Society of Civil Engineers deem it wise to establish some form of employment clearing house, it would immediately follow that similar action by the other engineering societies would, for the same reason, be equally advisable, thus rendering desirable a joint action which would increase the scope of the undertaking and tend toward economy in its management.

"Third.—If a co-operative engineering employment agency were to be formed, it could probably best be handled by a separate management and have separate headquarters from those now maintained by the existing societies, being supported by contributions from all societies participating, and subject to their control.

"In view of the foregoing, it is recommended that your present Committee be discharged and that, to inaugurate the movement, its report be referred either to a joint committee to be formed by the four founder societies, the membership of which should perhaps be composed of the secretaries of those societies, or to the Engineering Council, the participation in which is now being considered by your Board.

"This recommendation is made with the hope that early co-operative action with the other societies may result in the outline at the next Annual Convention of a joint action that will inure to the immediate personal benefit of a large portion of the Society's membership, as well as to the ultimate increase of the Society's usefulness to the profession at large.

"All of which is respectfully submitted.

"F. H. Newell, *Chairman,*
"H. S. Crocker,
"E. N. Layfield."

**Resolutions Relating to Universal Military Training and Service**

The following resolutions were unanimously adopted by the Board of Direction at its quarterly meeting of April 18th, 1917:

"*Whereas,* a state of war, between our Country and a great foreign power now exists, and

"*Whereas,* all previous experience of our own, no less than that of other nations, has demonstrated the danger, as well as insufficiency, of relying on volunteer armies and navies for the national defense; be it

"*Resolved:* That Congress be petitioned to pass at once a bill providing for universal military training and service, which we hold to be the only proper, democratic, and efficient way for creating the public defense; and

"*Be it further resolved:* That a copy of these resolutions be transmitted to the President of the United States and to each Senator and Member of the House of Representatives."

## Contingent Fees

The following correspondence, relating to the question of contingent fees, has been ordered printed by the Board of Direction for the information of the membership.

"January 3d, 1917.

"Mr. Charles Warren Hunt, *Secretary*,

"My dear Mr. Hunt: A case has arisen in our practice in which the claimants in a law suit have had their entire possessions wiped off the earth by the failure of a dam above their farms and orchards. They come to us and say, 'All our worldly possessions have been lost to us, and we want you to testify as to the design and construction of the dam whose failure caused our loss. We can pay you only a small retainer in cash, and will pay the remainder of your charges by giving you a small percentage of the award if we get a verdict in our favor. If the verdict is against us we cannot pay you anything more.'

"I know that engineers frequently take pay for their services as a percentage on the cost of a piece of work, and the engineer is trusted to keep the cost of the work as low as possible for his client, although it would be to his interest to make it as high as possible.

"Is this an analogous case, or would not an engineer in this case seem to be tempted to become a special advocate?

"It seems perfectly proper for a lawyer to accept a contingent fee, but whether it is so for an engineer is the question on which I should like to have your views. As all the members of our firm are members of the American Society of Civil Engineers, we believe that you will kindly favor us with your opinion on the subject. If you are in doubt yourself you may be able to lay the matter before one or two of the eminent engineers who call at your office from time to time.

"With kind regards, I am

"Yours sincerely,

"J. H. Quinton."

The Board of Direction referred the matter to Clemens Herschel, Past-President, Am. Soc. C. E., and the following is his report:

"New York, N. Y., January 22d, 1917.

"To the Board of Direction,

"Gentlemen:—On the letter referred to me for report, from Messrs. Quinton, Code and Hill, of Los Angeles, Cal., of January 3, 1917, I have to say:

"Lawyers in good standing, and jealous to maintain their good name, do not, as I understand it (but as stated in this letter), consider it 'perfectly proper to accept a contingent fee', in the ordinary course of their practice; still less to make agreements involving such acceptance.

"They occasionally accept such fees, on possibly a tacit understanding that such reward will be offered them, in the case of poor people, palpably injured in their rights or body, who without aid of this nature would be obliged to suffer without remedy.

"The letter on which a report is asked may be viewed specifically; or as it raises a general question.

"Specifically, the answer should be, in my opinion, that the Code of Ethics deals with the question asked, in paragraph numbered 1, and parties should govern themselves accordingly.

"To the Board, I will expatiate on this proposed reply, by pointing out, that an expert witness is frequently asked on cross-examination to state what pay he is getting. His influence with a jury and very likely also with any tribunal, would presumably be injured or gone, if he testified that a contingent fee had been stipulated. And paragraph 1, just referred to, declares that it is incumbent on members not 'to accept any remuneration other than his *stated* charges'.

"Passing to the general question or questions raised by the letter reviewed, it is evident that the whole broad question of the employment of expert witnesses, as witnesses, is brought into consideration.

"As the undersigned wrote at length on this subject in an essay printed by the Boston Bar Association, more than 30 years ago, he will content himself with saying, that all students of the subject are agreed that instead of expert witnesses, there should, in trials of the kind considered, be expert assistants to the presiding judge; as is the case in all countries of the earth, so far as he knows, except Great Britain and the United States of America.

"In the last named, the expert witness mode of making use of the services of experts in the conduct of judicial inquiries was begun in 1782, by the appearance on the witness stand (to give thence an opinion, not a statement of fact), of John Smeaton, the man who first in English speaking countries called himself a *Civil* Engineer, the builder of the noted Eddystone Lighthouse (see the leading case of Folkes *vs.* Chadd); and has been going from bad to worse, ever since.

"All of which is respectfully submitted.

"CLEMENS HERSCHEL."

"JANUARY 31, 1917.

"MR. CHAS. WARREN HUNT, *Sec'y.*

"MR. DEAR MR. HUNT: I have received both your letters on the subject of ethics. Mr. Herschel's letter, of which you kindly sent me a copy, is quite enlightening on the subject, and we will govern ourselves accordingly. I thank you very much for having brought this matter before the Society, and with kind regards, I remain,

"Yours very truly,

"J. H. QUINTON."

### Economy in the Food Resources of the Country

"WASHINGTON, D. C., April 14, 1917.

"TO COMMERCIAL ORGANIZATIONS:

"The war in which we are now engaged is a war of economic resources. It is absolutely essential to the successful prosecution of the war that every one make his or her contribution to the economic

welfare of the country. The production of food is a vital and present duty resting on every man and woman who can help it along. Without food workmen can not work, nor can armies fight.

"The food supply of the country must be increased, and I urge upon you to co-operate in every way with the Department of Agriculture in its campaign to increase the crops of the country. Will you not take this subject up at once with the membership of your association, pointing out to them the needs of the situation and urge immediate action? I suggest that every organization should have a committee on the production of foodstuffs and that the assistance of women's organizations be enlisted in the campaign.

"I need not point out that the planting season is at hand, and that any action to be effective must be taken at the earliest possible moment. In the United States, as in the warring countries of Europe, the effects of this struggle will be felt by every one, and economic preparedness will greatly lessen the burden that we must carry.

"Very truly yours,

"WILLIAM C. REDFIELD,

*"Secretary."*

## Proposed Bill for the Licensing of Engineers Before the State Legislature of California

The San Diego Association of Members of the American Society of Civil Engineers has recently taken action with reference to a Bill, relative to the licensing of architects, introduced in the State Legislature of California. A copy of the bill, and the resolution unanimously adopted by the Association are printed herewith.

AN ACT PROVIDING FOR PUBLIC SAFETY IN BUILDINGS WITHIN THE STATE OF CALIFORNIA BY REGULATING THE CONSTRUCTION, ALTERATION AND REPAIR OF SAME, AND TO PROVIDE PENALTIES FOR THE VIOLATION THEREOF.

*The people of the State of California do enact as follows:*

SECTION 1. Within sixty days from and after the passage of this act, the governor of the state shall appoint a board of seven members, which board shall be known as the state board of architecture. Until the appointment and qualification of a majority of the board herein provided the state board of architecture as organized under "An act to regulate the practice of architecture, chapter two hundred twelve, approved March 23, 1901, and approved March 26, 1903," shall constitute the said board. It shall be the duty of the state board of architecture to enforce all provisions of this act.

The membership of this board shall be composed as follows: All members shall be residents of the State of California; one member

shall hold at least a bachelor's degree in civil engineering from a recognized school of engineering, college or university, and shall have been engaged in continuous practice or teaching of the profession of civil engineering for a period of not less than ten years; six members shall hold certificates to practice architecture in the State of California. Three of the members holding certificates to practice architecture shall be residents of that portion of the state north of the northerly line of the county of San Luis Obispo, the county of Kern and the county of San Bernardino, hereinafter known as the northern district; three members holding certificates to practice architecture shall be residents of that portion of the state south of the northerly line of the county of San Luis Obispo, county of Kern and the county of San Bernardino, hereinafter known as the southern district.

In making the original appointments to the board, the term of office of the member qualified in civil engineering shall be three years. The term of office of one certificated member from the northern district and one from the southern district shall be two years each. The term of office of one certificated member from the northern district and one from the southern district shall be three years each. The term of office of one certificated member from the northern district and one from the southern district shall be four years each. After the original appointments the term of office of each member of the board shall be four years, or until a successor shall have been duly appointed and shall have qualified. Should any member of said board change residence from one district to the other during his term of office, said office shall become vacant. The governor shall have power to remove from office any member of the board for neglect of duty in the enforcement of this act, or for any cause which in his judgment renders such member incompetent to serve on said board. In the event of any vacancy occurring in the membership of the board, in any manner other than by expiration of the term herein set forth, the governor shall fill said vacancy by an appointment for the unexpired term. All members of the board, before entering upon the discharge of the duties of their offices as herein set forth, shall subscribe to and file with the secretary of state the constitutional oath of office.

SEC. 2. The board, within ninety days from and after the passage of this act, shall meet and elect from its membership a president and a vice-president, one of whom shall be from the northern district and one from the southern district; and a secretary, and an assistant secretary, one of whom shall be from the northern district and one from the southern district, who shall act as treasurer and assistant treasurer respectively. The term of office of such officers shall be two years each; the president shall not be eligible to reelection, and shall be selected alternately from the northern and the southern districts.

The board shall adopt all necessary rules, regulations and by-laws, not inconsistent with this act and the constitution and laws of this state or of the United States, requisite to the exercise of its powers and duties as in this act provided. The board shall adopt a seal, of

which the secretary shall have the care and custody. The secretary shall keep a correct record of the proceedings of the board, of fees received and moneys disbursed, which record shall be open to public examination at all times. Four members of the board shall constitute a quorum for the transaction of business.

Regular meetings of the board shall be held on the last Tuesday in April of each year in San Francisco, and on the last Tuesday in October of each year in Los Angeles, at each of which meetings examinations of applicants for "certificates to practice architecture" shall be held. Special meetings to transact any business that may come before the board shall be called by the president within thirty days after the written request so to do from not less than three members of said board, and as prescribed in the by-laws adopted by said board.

An annual report of the work and proceedings of the board, embodying the report of the secretary as treasurer, together with a complete directory giving the names and addresses of all persons who hold unrevoked "certificates to practice architecture" in the State of California, shall be made by said board to the governor of the state, and a copy of said report shall be mailed to each person who holds a "certificate to practice architecture" in the State of California.

SEC. 3. Each member of the board shall serve without compensation for his services, but the board may incur such expenses as it shall deem to be necessary in the enforcement of this act, and the members of the board shall be reimbursed for the expenses incurred by them in the performance of their duties under this act. Expenses of the board shall be paid out of the fees collected and retained by the board, as in this act provided. At the end of each fiscal year any excess of fees received over moneys disbursed, after a working cash balance of one thousand five hundred dollars has been retained, shall be paid by the board to the state treasurer, to be retained by the state. All moneys and assets of the district and state boards of architecture existing at the time of the passage of this act shall become the property of the state board of architecture as organized under this act, and the secretary of each existing board shall turn such assets over to the secretary of the new board, together with a complete report and accounting of all such moneys and other assets.

SEC. 4. For the purpose of this act, unless it should be apparent from their context that they have a different meaning, words used in the singular include the plural, and the plural, the singular; words used in the present tense include the future; words used in the masculine gender include the feminine, and the feminine, the masculine; and "shall" as used shall be deemed to be mandatory. "The board" shall be deemed to be the state board of architecture as organized under this act.

SEC. 5. It shall be unlawful for any person, firm or corporation to construct, erect, alter, add to, repair or reconstruct any building, or portion thereof, which is, or is designed or intended to be occupied in whole or in part by human beings for the purposes of living, sleeping or assemblage, for manufacturing or repair, whether by machinery or hand, for the sale and storage of goods or merchandise, or for the pur-

suit of vocations, whenever such construction, erection, alteration, addition to, repair or reconstruction involves or affects the stability, strength or safety of such building, unless such construction, alteration, erection, addition to, repair or reconstruction shall have been designed, specified and supervised or superintended by a person who has complied with all the requirements of this act, and who holds a "certificate to practice architecture" as herein provided, or by a person who is under the direct control and supervision of and is acting for and is responsible to a person holding the "certificate to practice architecture" as in this act provided; *provided, however,* that nothing in this section shall make it unlawful for any person to construct, erect, alter, add to, repair or reconstruct any such building or portion thereof which is, or is designed or intended to be occupied or used as a home by not more than one family for living and sleeping purposes.

It shall be unlawful for any person, firm or corporation, or for any person who is in any manner connected with such firm or corporation, either directly or indirectly, to use or attempt to use any fraudulent methods or evidence to obtain an examination required by this act; to use, obtain, or offer or attempt to use or obtain a certificate required by this act in a fraudulent manner; to use or attempt to use any certificate, license, registration or similar paper or document, whether genuine or false, in any manner for the purposes set forth in this act, except for the certificate as provided for in this act; or to use or maintain or attempt to use or maintain in any sign or advertisement, or in stamping, signing, or labeling drawings, specifications, contracts, correspondence, or advertising of any character, the words "architect" or "architects," or any abbreviation of same, or any combination of words, or device of which such word forms a part, with the object or result, either intentional or unintentional of designating such person, firm or corporation as "architect" or "architects" and qualified to perform the lawful functions of a person who holds a "certificate to practice architecture" under this act.

Any person, firm or corporation, violating any of the provisions of this section of this act set forth shall be deemed guilty of a misdemeanor, and upon conviction thereof shall be punished by a fine not less than fifty dollars and not exceeding five hundred dollars, or by imprisonment in a county jail not exceeding six months, or by both such fine and imprisonment; and in addition to the penalty therefor shall be liable for all costs, expense and disbursements by this act provided, which costs, expense and disbursements shall be fixed by the court having jurisdiction of the matter.

SEC. 6. Any person who has given to the board satisfactory proof that he is not less than twenty-one years of age, that he is of good moral character, and that for a period of not less than five years he has been continuously engaged in designing, making drawings or specifications, supervising or superintending the erection of buildings (not to exceed one of the five years may have been spent in travel for the purpose of study of such subjects), either as a practicing architect not within the State of California or in the employ of such a person whose residence is not within said state, or in the employ of a person or persons holding a "certificate to practice architecture" within the State of California, shall be entitled to an examination for a "certificate to

practice architecture" before and by said board; and upon payment by such person to the board of a fee of fifty dollars, such fee to be retained by the board, the board shall examine such person at its next following regular examination of applicants for certificates.

A person granted the examination by the board who shall fail to pass such examination to the satisfaction of the board, shall not be eligible for re-examination for a period of at least six months after such failure; and if such person has failed to pass the examination in more than one-half of the subjects required in such examination, upon re-examination such person shall pay a fee of fifty dollars, which fee shall be retained by the board. If such person has failed to pass the examination in less than one-half of the subjects required in such examination, upon re-examination such person shall pay a fee of twenty-five dollars, which fee shall be retained by the board. Whenever any person examined by the board shall have passed such examination to the satisfaction of the board, or whenever any person, whose place of residence and principal place of business are not within the State of California, shall have had issued to him in such state in which he resides a license or certificate of qualification wherein, in the judgment of the board, the qualifications of the person and the standard of examination for such license or certification are not less in any of their requirements than those prescribed by this act, and whenever such person shall have paid to the secretary of the board the annual current license fee, the secretary of the board shall issue to such person a certificate, signed by the president and secretary, sealed with the seal of the board and directed to the secretary of state, certifying that the person therein named has passed an examination satisfactory to the board, and that such person is entitled to a "certificate to practice architecture" within the State of California, in accordance with the provisions of this act.

To the person presenting such certificate from the board, and upon payment by said person to the secretary of state of a fee of ten dollars, the secretary of state shall at once issue to the person therein named a "certificate to practice architecture" in this state in accordance with the provisions of this act, which certificate shall contain the full name of the person, his birthplace and age, the date of the issuance of the certificate, and shall state the fact of his having passed an examination satisfactory to the state board of architecture, and the date of such examination. The secretary of the board shall transmit directly to the secretary of state the full record and report of the examination, and the secretary of state shall keep same on file in his office as public records, together with a proper index and record thereof. All certificates heretofore issued by the state board of architecture under the provisions of the act entitled "An act to regulate the practice of architecture, chapter two hundred twelve, approved March 23, 1901, and approved March 26, 1903," shall have the same virtue, force and effect and shall be henceforth subject to the provisions of this act.

SEC. 7. The requirements of the examination, as in this section provided to be given by the board to determine the qualifications of the person for "certificate to practice architecture," are hereby fixed to be the minimum requirements for examination by the board, and such requirements may be increased by the action of the board, but

shall never be waived by the board, except when the person applying for such examination, whose place of residence and principal place of business are not within the State of California, and where in such place of business a license or certificate of qualification is not required by law, is and has been for a period of not less than ten years previous to said time of application, engaged in the active practice of architecture, the board, upon satisfactory proof of ability and qualification of said applicant, may waive the written examination prescribed by this act. The applicant shall be examined by the board as to the following:

(a) His practical experience, its extent and responsibility, his technical knowledge of materials, their strength and use in practical construction; and his ability to compute mathematically the strength and stresses in materials and structures, and to design a building or structure, or any portion thereof, so as to insure inherent stability and strength in all its parts, and to meet the contingencies and problems of construction and public safety that arise in the erection of buildings or structures, or portions thereof. This portion of the examination shall be in writing, supplemented orally, and shall be comprehensive, in order to insure that the applicant has a solid theoretical understanding and a working knowledge of the principles and mathematics involved in the computing of all stresses and strains in the mechanics of building operations.

(b) His theoretical and practical knowledge of sanitation as applied to buildings, and his ability to design plumbing systems therein.

(c) His knowledge of the theory and design of heating and ventilating of buildings, and his practical understanding of the various systems in use.

(d) His knowledge of architectural history, terms, forms and design.

(e) His general education, together with his character and fitness for a certificate.

SEC. 8. Every person to whom a "certificate to practice architecture" has been issued in accordance with the provisions of this act shall have a seal, the impression of which must contain the name of the person so certificated, his place of business and the words, "certificated to practice architecture," or if such person is doing business as a member of a firm or corporation, the firm or corporation shall have a firm or corporate seal, the impression of which must contain the separate name of each of the persons certificated, followed by the words "certificated to practice architecture," and the place of business of such firm or corporation. Every such person, firm or corporation shall stamp with such seal every drawing, plan, specification and contract prepared by said person, firm or corporation. A failure to so do shall be sufficient cause for the revocation and cancellation of said certificate as in this section provided.

Every person to whom a "certificate to practice architecture" has been issued, in accordance with this act, shall have his certificate recorded in the office of the county recorder in the county in this state in which the holder thereof resides; shall pay to the recorder the same fee therefor as is charged for the recording of deeds; and shall notify the secretary of the board in writing of such recording, who shall acknowledge same under seal of the board. A failure to have his cer-

tificate so recorded shall be deemed sufficient cause for revocation and cancellation of such certificate, as in this section provided.

Every person to whom a "certificate to practice architecture" has been issued, in accordance with this act, shall pay an annual license fee of five dollars to the state board of architecture, such fee to be retained by the board. Such fee shall be payable in advance on the first Monday in January of each year. When paid the secretary of the board shall issue to each such person a receipt signed by the president and secretary under the seal of the board. If any such person shall fail, neglect or refuse to pay such annual license fee on or before the first Monday in April of each year, said fee shall be delinquent, and the certificate of such person shall thereupon become subject to revocation and cancellation.

Each "certificate to practice architecture" issued in accordance with the provisions of this act shall remain in full force until revoked and canceled for cause, as provided for in this section. A "certificate to practice architecture" may be revoked and canceled for failure to stamp contracts, drawings, plans or specifications; for stamping or signing such contracts, drawings, plans or specifications in any manner to certify to, or to appear to certify to, any such contracts, drawings, plans, or specifications, when any of these were not made by him or under his direct supervision as in this act provided; for failure to record; for failure to pay annual license fees; for conviction of fraud, misdemeanor under this act, felony, incompetency, gross carelessness or dishonest practice.

It shall be the duty of the state board of architecture, whenever charges in writing have been filed with the board and signed with the name of the person bringing such charges, against any person holding a "certificate to practice architecture" in the State of California, charging such person with any of the causes of revocation and cancellation of the "certificate to practice architecture," as in this act provided, to notify such person in writing of the nature of the charges, and to cite him to appear before said board to show cause why such certificate should not be revoked and canceled. Such notification and citation shall be given not less than thirty days before the time set for the hearing, and the delivery of such notice and citation to the place of business or last known address of such person shall be deemed sufficient for the purpose of this notification and citation. Should such person so cited fail, neglect or refuse to appear at the time set by the board for the hearing, this shall be deemed evidence of guilt, and the board shall thereupon revoke and cancel the certificate of such person. The board shall investigate fully the charges against such person at the time set for the hearing, and the person charged shall be given opportunity to be heard in his own defense or to be represented by counsel.

After such hearing, if, in the judgment of not less than three members of the board, the charges against such person have been proven, the secretary of the board shall issue to the secretary of state a certificate revoking the "certificate to practice architecture" of such person, and the secretary of state shall then cancel such certificate.

Upon the cancellation of such certificate, it shall be the duty of the secretary of the board to give notice of such cancellation to the county recorder of that county in the state in which said certificate

has been recorded, whereupon the recorder shall mark the certificate recorded in his office "canceled." At the expiration of not less than six months after said cancellation, the person whose "certificate to practice architecture" has been canceled may apply to the board for a new certificate, and the board may grant same upon the payment of every fee required in this act for any applicant for examination (but shall never grant such certificate when the same has been revoked for incompetency or gross carelessness), without requiring such person to pass the examination prescribed by this act.

SEC. 9. If any such section, subsection, sentence, clause or phrase of this act is for any reason held to be unconstitutional, such decision shall not affect the validity of the remaining portions of this act. The legislature hereby declares that it would have passed this act, and each section, subsection, sentence, clause and phrase thereof, irrespective of the fact that any one or more sections, subsections, sentences, clauses or phrases be declared unconstitutional.

SEC. 10. An act entitled "An act to regulate the practice of architecture," approved March 23, 1901, and approved March 26, 1903, is hereby repealed.

**Resolution Adopted by San Diego Association**

"*Whereas,* There is now pending before the State Legislature of the State of California, Assembly Bill No. 1126, which is an act 'Providing for Public Safety in Buildings within the State of California by Regulating the Construction, Alteration and Repair of same, and to Provide Penalties for the Violators thereof, and,—

"*Whereas,* the San Diego Association of the American Society of Civil Engineers are firmly convinced that this bill will not provide public safety; that it is not along the line of public policy; that it entails unnecessary burdens and expense; that it creates a monopoly and discriminates in that the Engineer, who by education, training, experience and practice, is especially and peculiarly qualified and adapted to design, construct, alter and repair buildings and structures, and,—

"*Whereas,* if this bill becomes a law it will be necessary that every building, excepting a house wherein but one family is to reside, shall be designed, constructed, repaired or altered by a licensed architect, as the same is defined in the bill, even to the designing, construction, repairing or alteration of a chicken house, garage, shed, etc.

"*Therefore, be it Resolved*; That the San Diego Association of the American Society of Civil Engineers is unanimously and unalterably opposed to the enactment of this bill and that the said San Diego Association shall use such means and methods as are fit and proper to accomplish the defeat and prohibit the enactment of same into a law.

"*Unanimously Adopted* by the San Diego Association of the American Society of Civil Engineers in regular session assembled at San Diego, California, on March 6th, 1917.

"W. J. GOUGH, President.
"SAN DIEGO ASSOCIATION,
"AMERICAN SOCIETY OF CIVIL ENGINEERS.
"Attest: P. R. WATSON,
"*Secy. Pro tem.*"

## The Engineering Council

The United Engineering Society has adopted the following amendments to its By-Laws in order to provide for convenient co-operation between the four Founder Societies, for the proper consideration of questions of general interest to engineers and to the public and to provide the means for united action upon questions of common concern to engineers.

"The United Engineering Society shall, in addition to its other activities and entirely separate therefrom, establish, and maintain a department to be known as

### Engineering Council*

"Each Founder Society shall have five representatives upon the Council and such representatives shall be designated by the governing body of such Society.

"The United Engineering Society shall have four representatives on the Engineering Council who shall be chosen by its Board of Trustees; such representatives to be men not already designated by a Founder Society.

"The Council may speak authoritatively for all member societies on all public questions of a common interest or concern to engineers, unless objection be made by a majority of the representatives present of one of the Founder Societies or by one-fourth of the representatives present and voting. It shall defer any action which is opposed as aforesaid; such suspended action shall be referred to the governing bodies of the societies whose representatives have protested, for an expression of opinion of a veto upon further action, and may then be reconsidered if such societies have agreed.

"The Council shall elect annually (at its annual meeting) from the representatives a Chairman, and two Vice-Chairmen; also a Secretary who may or may not be a representative.

"The expenses of the Engineering Council shall be disbursed out of such funds as the Trustees of United Engineering Society shall from time to time provide for that purpose.

"The Engineering Council shall hold an annual meeting in New York in the month of February and such other meetings as the Council may decide.

"A quorum shall consist of one-third of the total number of representatives upon the Council.

"An Executive Committee to serve one year shall be constituted of six members, of which the Chairman, and two Vice-Chairmen shall be members. It shall transact necessary business between meetings of the Council, but shall take no action binding the Council upon debatable questions of engineering concern.

"The Council shall have authority to make rules for its own guidance, not inconsistent with these by-laws.

"The Trustees of the United Engineering Society may elect to membership in the Engineering Council other National Engineering

* See Report of Board of Direction, p. 312.

**or Technical Societies, under such rules as the Council prescribes provided their nomination and said rules have the unanimous approval of the governing bodies of the four Founder Societies.**

**"The Council shall keep a record of its proceedings and transmit, after each meeting, a copy of the same to the Board of Trustees of the United Engineering Society."**

## ANNOUNCEMENTS

**The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.**

## FUTURE MEETINGS

**September 5th, 1917.—8.30 P. M.**—A regular business meeting will be held, and a paper by Floyd A. Nagler, Jun. Am. Soc. C. E., entitled "Obstruction of Bridge Piers to the Flow of Water", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

### ANNUAL CONVENTION ABANDONED

The following is a copy of a circular issued to the membership:

Under date of May 3d there was issued to all members a circular concerning the Forty-ninth Annual Convention of the Society, to be held at St. Paul and Minneapolis, Minn., June 12th to 15th, 1917. All information concerning the Convention, general programme, etc., was given in some detail.

Up to that date the preparations for holding the Convention had gone on, although there was some doubt as to whether there would be an adequate attendance. On May 7th the War Department authorized the formation of nine Volunteer Engineer Regiments for immediate service in France, and this action has forced a realization of conditions affecting the members of this Society.

In this emergency the Executive Committee, which is authorized to act for the Board of Direction on urgent questions, was called together.

After careful discussion of the situation this Committee decided unanimously that the wise course to pursue was to abandon the Convention of 1917.

The following resolutions were unanimously adopted:

**"Resolved: That in view of the present condition of war, which involves the absence of many members of the Society in the service of their country, that the Annual Convention, which it had been decided to hold at St. Paul and Minneapolis, June 12-15 (inclusive), be abandoned, and that the membership be notified at once by a circular giving the reasons governing the Executive Committee in its action."**

**"Resolved: That it is the sense of the Executive Committee that the next Annual Convention of the Society should be held at St. Paul and Minneapolis, provided it is the desire of the membership in that locality, and that thereafter the rotation of the place for holding the Convention in the various Districts be maintained as heretofore arranged."**

In carrying out the foregoing resolutions, in addition to the reason given therein, the replies already received to the Convention circular, as well as numerous interviews with members of the Society in the East, make it practically certain that, if the Convention were held as scheduled, the outside attendance would be very small. Several of those who have stated that they would not attend have indicated that existing conditions prevented them. Quite a number of other organizations who hold Annual Conventions, and whose meetings had been scheduled and prepared for, have taken similar action.

By order of the Executive Committee.

George H. Pegram,
*President.*

May 10th, 1917.

Chas. Warren Hunt,
*Secretary.*

## SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions.*

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

**Hereafter, therefore, requests for searches should be addressed to the Librarian, United Engineering Society, 29 West 39th Street, New York City.**

## PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

## LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

**San Francisco Association, Organized 1905.**

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August,

October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

**Colorado Association, Organized 1908.**

Thomas W. Jaycox, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

**March 10th, 1917.**—The meeting was called to order at the Denver Athletic Club; President Jaycox in the chair; L. R. Hinman, Secretary; and present, also, 20 members and 3 guests.

The minutes of the meeting of February 10th, 1917, were read and approved.

The resignation, as a member of the Association, of Mr. I. O. Thorley, now a resident of Detroit, Mich., was presented and accepted.

The Secretary read a letter from the St. Louis Association in the matter of proposed amendments to the Constitution of the Society relative to representation on the Board of Direction. On motion, duly seconded, the reading of the amendments, as presented by the St. Louis Association, was deferred.

President Jaycox addressed the meeting informally on the work done by the Miami Conservancy District for flood protection in the Miami Valley, in Ohio, illustrating his remarks with lantern slides.

A paper by John E. Hayes, Chief Engineer of the Farmers Reservoir and Irrigation Company, on "The Construction of Standley Dam and Recent Slides in the Embankment", was presented by the author, who afterward explained the paper with the aid of lantern slides.

The subject was discussed by Messrs. C. W. Comstock and L. E. Bishop, the former of whom also illustrated his remarks with lantern slides.

Messrs. Ridgway, Black, and Norton were appointed the Entertainment Committee for the April Meeting.

On motion, duly seconded, a vote of thanks was extended to Mr. Hayes for his paper.

Adjourned.

**Atlanta Association, Organized 1912.**

Paul H. Norcross, President; Thomas P. Branch, Secretary-Treasurer, Georgia School of Technology, Atlanta, Ga.

The Association holds its meetings at the University Club, Atlanta, Ga. Regular monthly luncheon meetings are held to which visiting members of the Society are always welcome.

**Baltimore Association, Organized 1914.**

H. D. Bush, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

(Abstract of Minutes of Meeting)

**March 21st, 1917.**—The meeting was called to order at 8.35 P. M., at the Engineers' Club; President Bush in the chair; C. J. Tilden, Secretary; and present, also, 24 members.

The minutes of the meetings of the Association of May 3d, 1916, and of the Board of Directors of March 9th, 1917, were read and approved.

Mr. W. D. Janney explained the objects and advantages of the Associated Technical Societies of Baltimore, of which the Association became a member by action of its Board of Directors on March 9th, 1917.

The Secretary read the proposed amendments to the Constitution of the Association, as submitted at the March 9th meeting of the Board of Directors, and reported that it had been sent to the Board of Direction of the Society for its approval.

On motion, duly seconded, it was decided that President Bush appoint a committee of three to arrange for a dinner to be held before the Annual Meeting of the Association in May. President Bush appointed Messrs. Greiner, Whitman, and Warren as such Committee. It was further decided that this Committee discuss the question of monthly luncheons for the Association, and take such action in that regard as deemed by it desirable.

The Secretary read a letter from Mr. Charles Warren Hunt, Secretary of the Society, announcing the appointment of a committee of the Board of Direction to study and report on the relations of Local Associations to the Society. The subject was discussed briefly by Messrs. Janney, Greiner, Whitman, and Pitts, and, on motion, duly seconded, the President was authorized to appoint a committee to prepare a definite plan relative to this matter, to be presented at the Annual Meeting of the Association on May 2d, 1917. President Bush subsequently appointed Messrs. Janney, Pagon, and Tilden as such Committee.

A communication from the St. Louis Association relative to the proposed change in the Constitution of the Society, was read by the Secretary. The subject was discussed briefly by Messrs. L. F. Smith, Whitman, Pitts, and Janney, and, on motion, duly seconded, it was resolved that it is the sentiment of the Association that the Constitution of the Society should be allowed to stand as regards the requirements concerning Directors and that notice to that effect be sent to the Secretary of the Society.

Mr. R. K. Compton introduced the subject of the Engineer Section of the Officers' Reserve Corps, explaining the provisions of General

Orders 32, providing for the creation and maintenance of this Corps. A general discussion of the subject by the members followed, and President Bush read an editorial from a recent number of the *Boston News Bureau*.

Adjourned.

**Cleveland Association, Organized 1914.**

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

**Detroit Association. Organized 1916.**

T. A. Leisen, President; Secretary, Clarence W. Hubbell, 2334 Dime Bank Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

**District of Columbia Association, Organized 1916.**

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

**Duluth Association, Organized 1917.**

Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held monthly. The time and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

The Annual Meeting is held on the third Monday of May in each year.

**Illinois Association, Organized 1916.**

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

**Louisiana Association, Organized 1914.**

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

**Nebraska Association, Organized 1917.**

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

It is probable that frequent luncheons will be held in Omaha, in addition to the monthly meetings, at which visiting members will be welcomed. The place of meeting may be ascertained by communicating with the Secretary.

(Abstract of Minutes of Meeting)

**April 10th, 1917.**—The meeting was called to order at the Lincoln Hotel, Lincoln, Nebr., at 7.40 P. M.; President Darrow in the chair; Homer V. Knouse, Secretary; and present, also, 10 members and 9 guests.

The Secretary read a letter from T. C. Desmond, Assoc. M. Am. Soc. C. E., in regard to the Roosevelt Division for service in France, and the announcement of the next meeting of the Colorado Association was also read.

At the request of Mr. C. E. Mickey, Mr. Herbert S. Crocker, a Director of the Society, discussed various matters of interest now before the Society for consideration and action.

The meeting then adjourned to the University of Nebraska, and Mr. Crocker presented a paper entitled "Certain Trying Features of the Construction of Reinforced Concrete Viaducts," which he illustrated with lantern slides of work on the Colfax-Larimer Viaduct, recently constructed at Denver, Colo.

An expression of the appreciation of the Association for Mr. Crocker's address was made by President Darrow.

Adjourned.

**Northwestern Association, Organized 1914.**

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

**Philadelphia Association, Organized 1913.**

Samuel T. Wagner, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the first Monday in January, April, and October, the last being the Annual Meeting.

**Portland, Ore., Association, Organized 1913.**

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

**St. Louis Association, Organized 1914.**

J. A. Ockerson, President; Gurdon G. Black, Secretary-Treasurer, 34 East Grand Avenue, St. Louis, Mo.

The meetings of the Association are held at the Engineers' Club Auditorium. The Annual Meeting is held on the fourth Monday in November. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

**San Diego Association, Organized 1915.**

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

**Seattle Association, Organized 1913.**

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Arctic Club.

**Southern California Association, Organized 1914.**

H. Hawgood, President; Wilkie Woodard, Secretary, 435 Consolidated Realty Building, Los Angeles, Cal.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings, with banquet, at Hotel Clark, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

**Spokane Association, Organized 1914.**

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

**Texas Association, Organized 1913.**

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

**Utah Association, Organized 1916.**

E. C. La Rue, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

## PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

**American Institute of Electrical Engineers,** 25 West Thirty-ninth Street, New York City.

**American Institute of Mining Engineers,** 25 West Thirty-ninth Street, New York City.

**American Society of Mechanical Engineers,** 25 West Thirty-ninth Street, New York City.

**Architekten-Verein zu Berlin,** Wilhelmstrasse 92, Berlin W. 66, Germany.

**Associação dos Engenheiros Civis Portuguezes,** Lisbon, Portugal.

**Australasian Institute of Mining Engineers,** Melbourne, Victoria, Australia.

**Boston Society of Civil Engineers,** 715 Tremont Temple, Boston, Mass.

**Brooklyn Engineers' Club,** 117 Remsen Street, Brooklyn, N. Y.

**Canadian Society of Civil Engineers,** 176 Mansfield Street, Montreal, Que., Canada.

**Civil Engineers' Society of St. Paul,** St. Paul, Minn.

**Cleveland Engineering Society,** Chamber of Commerce Building, Cleveland, Ohio.

**Cleveland Institute of Engineers,** Middlesbrough, England.

**Dansk Ingeniorforening,** Amaliegade 38, Copenhagen, Denmark.

**Detroit Engineering Society,** 46 Grand River Avenue, West, Detroit, Mich.

**Engineers' and Architects' Club of Louisville,** 1412 Starks Building, Louisville, Ky.

**Engineers' Club of Baltimore,** 6 West Eager Street, Baltimore, Md.

**Engineers' Club of Kansas City,** E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

**Engineers' Club of Minneapolis,** 17 South Sixth Street, Minneapolis, Minn.

**Engineers' Club of Philadelphia,** 1317 Spruce Street, Philadelphia, Pa.

**Engineers' Club of St. Louis,** 3817 Olive Street, St. Louis, Mo.

**Engineers' Club of Toronto,** 96 King Street, West, Toronto, Ont., Canada.

**Engineers' Club of Trenton,** Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

**Engineers' Society of Northeastern Pennsylvania,** 415 Washington Avenue, Scranton, Pa.

**Engineers' Society of Pennsylvania,** 31 South Front Street, Harrisburg, Pa.

**Engineers' Society of Western Pennsylvania,** 568 Union Arcade Building, Pittsburgh, Pa.

**Institute of Marine Engineers,** The Minories, Tower Hill, London, E., England.

**Institution of Civil Engineers,** Great George Street, Westminster, S. W., London, England.

**Institution of Engineers of the River Plate,** Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

**Institution of Naval Architects,** 5 Adelphi Terrace, London, W. C., England.

**Junior Institution of Engineers,** 39 Victoria Street, Westminster, S. W., London, England.

**Koninklijk Instituut van Ingenieurs,** The Hague, The Netherlands.

**Louisiana Engineering Society,** State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

**Memphis Engineers' Club,** Memphis, Tenn.

**Midland Institute of Mining, Civil and Mechanical Engineers,** Sheffield, England.

**Montana Society of Engineers,** Butte, Mont.

**North of England Institute of Mining and Mechanical Engineers,** Newcastle-upon-Tyne, England.

**Oesterreichischer Ingenieur- und Architekten-Verein,** Eschenbachgasse 9, Vienna, Austria.

**Oregon Society of Civil Engineers,** Portland, Ore.

**Pacific Northwest Society of Engineers,** 803 Central Building, Seattle, Wash.

**Rochester Engineering Society,** Rochester, N. Y.

**Sachsischer Ingenieur- und Architekten-Verein,** Dresden, Germany.

**Sociedad Colombiana de Ingenieros,** Bogota, Colombia.

**Sociedad de Ingenieros del Peru,** Lima, Peru.

**Societe des Ingenieurs Civils de France,** 19 rue Blanche, Paris, France.

**Society of Engineers,** 17 Victoria Street, Westminster, S. W., London, England.

**Svenska Teknologforeningen,** Brunkebergstorg 18, Stockholm, Sweden.

**Tekniske Forening,** Vestre Boulevard 18–1, Copenhagen, Denmark.

**Vermont Society of Engineers,** George A. Reed, Secretary, Montpelier, Vt.

**Western Society of Engineers,** 1737 Monadnock Block, Chicago, Ill.

## ACCESSIONS TO THE LIBRARY*

(From April 4th to April 30th, 1917)

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* The Library of the Society having been discontinued, these accessions have been turned over to the United Engineering Society Library, 29 West 39th Street, New York City, to which future donations should be made. This list will not be published in the future.

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| U. S.-Naval Observatory. 1 bound vol. | Western Reserve Univ. 1 vol. |
| U. S. Steel Corporation. 1 pam. | Wyoming-State Geologist. 1 pam. |

## SUMMARY OF ACCESSIONS

(From April 4th to April 30th, 1917)

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## MEMBERSHIP

(From April 6th to May 3d, 1917)

### ADDITIONS

| MEMBERS | | Date of Membership. |
| --- | --- | --- |
| ALLEE, DAVID ARTHUR. Dept. Head, Gen. Elec. Co., Room 506, Gen. Elec. Co., Schenectady, N. Y. | | April 17, 1917 |
| BENNETT, JOHN WALTER FRINK. Cons. Engr. to Pres., Borough of The Bronx, 1882 Grand Concourse, New York City | | April 17, 1917 |
| BLACK, CLARENCE NEELLY. Chf. Engr., Gulf Pipe Line Co. and Gulf Production Co., P. O. Drawer 1791 (Res., 2204 Crawford St.), Houston, Tex. | Assoc. M.<br>M. | June 4, 1913<br>April 18, 1917 |
| CANNON, SYLVESTER QUAYLE. City Engr., 301 City and County Bldg., Salt Lake City, Utah | | Jan. 15, 1917 |
| CLINE, MCGARVEY. Vice-Pres., Florida Pine Co.; Engr., Commodores Point Terminal Co., 1427 Post St., Jacksonville, Fla. | | Jan. 15, 1917 |
| CUNNINGHAM, JOHN GEORGE LAWRENCE. Cons. Engr., Deer Lodge, Mont. | Assoc. M.<br>M. | Jan. 5, 1909<br>April 18, 1917 |
| FRENCH, CARSON GEYER. Cons. Structural Engr., 1836 Euclid Ave., Cleveland, Ohio | Assoc. M.<br>M. | May 28, 1912<br>April 18, 1917 |
| GARRETT, JAMES MADISON. 120 Catoma St., Montgomery, Ala. | | April 17, 1917 |
| GIBSON, NORMAN ROTHWELL. Asst. Chf. Engr., The Ontario Power Co. of Niagara Falls, 107 Culp St., Niagara Falls, Ont., Canada | | April 17, 1917 |
| GROVE, WILLIAM GARRETT. Asst. Engr., Am. Bridge Co., 30 Church St., New York City | Assoc. M.<br>M. | April 18, 1916<br>April 18, 1917 |
| HALE, HERBERT MILLER. Asst. Mgr., Holbrook, Cabot & Rollins Corporation, 52 Vanderbilt Ave., New York City | Jun.<br>Assoc. M.<br>M. | Nov. 1, 1904<br>Mar. 5, 1912<br>April 18, 1917 |
| HOLLAND, CLIFFORD MILBURN. Div. Engr., Public Service Comm., East River Tunnels, 933 East 22d St., Brooklyn, N. Y. | Assoc. M.<br>M. | Oct. 29, 1912<br>April 18, 1917 |
| JOHNSTON, JAMES HOUSTOUN. Cons. Engr., Western & Atlantic R. R. Comm., State Capitol, Atlanta, Ga. | | Mar. 13, 1917 |
| LUCAS, GEORGE LATIMORE. Gen. Insp. of Materials, Public Service Comm., First Dist., 120 Broadway, New York City | Assoc. M.<br>M. | Dec. 7, 1904<br>Jan. 16, 1917 |
| MCLANE, GLENWOOD LYLE. City Engr., Hutchinson, Kans. | | April 17, 1917 |
| NOYES, EDWARD NEWTON. Engr. (Myers & Noyes), 405 Juanita Bldg., Dallas, Tex. | Assoc. M.<br>M. | April 2, 1912<br>Jan. 16, 1917 |

MEMBERS (*Continued*)

| | | Date of Membership. |
|---|---|---|
| **Randall, Frank Alfred.** Civ. and Structural Engr. (Randall & Warner). 79 West Monroe St., Chicago, Ill. | Assoc. M.<br>M. | Dec. 5, 1911<br>April 18, 1917 |
| **Rayburn, John Matthew.** Civ. and Min. Engr., 1115 House Bldg., Pittsburgh, Pa. | | April 17, 1917 |
| **Varney, Henry Aldis.** Town Engr., Room 15, Town Hall, Brookline, Mass. | | April 17, 1917 |
| **Warren, Horace Prettyman.** Eng. Representative in United States for Alaska Eng. Comm., Seattle, Wash. | Assoc. M.<br>M. | Jan. 3, 1911<br>April 18, 1917 |
| **Weiss, Andrew.** Project Mgr., U. S. Reclamation Service, Mitchell, Nebr. | Assoc. M.<br>M. | Jan. 8, 1908<br>April 18, 1917 |
| **Wynn, Wesley Akers.** Engr., Bureau of Township Highways, State Highway Dept., P. O. Box 422, Harrisburg, Pa. | Assoc. M.<br>M. | April 4, 1911<br>April 18, 1917 |
| **Young, Charles Asa Dilts.** Asst. Engr., U. S. Engr. Dept., Box 1809, Seattle, Wash. | Assoc. M.<br>M. | Jan. 7, 1913<br>April 18, 1917 |

ASSOCIATE MEMBERS

| | | |
|---|---|---|
| **Anderson, Howard Royston.** Asst. Road Engr., Fayette County, Mt. Hope, W. Va. | | Mar. 13, 1917 |
| **Boesch, Clarence Edwin.** Charlotte, N. C. | | Nov. 28, 1916 |
| **Brennan, Joseph Lawrence.** Contr., 1711 University Ave., New York City | Jun.<br>Assoc. M. | Jan. 5, 1909<br>Mar. 13, 1917 |
| **Chamberlain, Joseph Jenks, Jr.** Asst. Engr., Bridge Dept., Cuyahoga County, 1452 East 135th St., Cleveland, Ohio | Jun.<br>Assoc. M. | Oct. 3, 1911<br>Mar. 13, 1917 |
| **Clark, Roy Ross.** Bridge and Structural Engr., 416 Selling Bldg., Portland, Ore. | | April 17, 1917 |
| **Congdon, Howard Wilbur.** Structural Engr., Providence Steel & Iron Co., Providence, R. I. | | April 17, 1917 |
| **Cotton, William Owen.** Chf. Engr. and Mgr., Idaho Irrig. Dist., 224 Salesbarry Earl Bldg., Idaho Falls, Idaho | Jun.<br>Assoc. M. | Jan. 17, 1916<br>April 17, 1917 |
| **Cykler, Emil Frank.** Chf. Engr., Lord-Young Eng. Co., Ltd., 2-A, Pantheon Bldg., Honolulu, Hawaii | Jun.<br>Assoc. M. | Oct. 31, 1911<br>Mar. 13, 1917 |
| **Farnsworth, Howard Richards.** U. S. Surv., Div. E, Gen. Land Office, Washington, D. C. | | April 17, 1917 |
| **Fraser, Robert Marshall.** Engr. of Constr., Rome Wire Co., Rome, N. Y. | | April 17, 1917 |
| **Galbreath, Albert Webster.** Asst. Engr., Vandalia R. R., 1314 Syndicate Trust Bldg., St. Louis, Mo. | | April 17, 1917 |

ASSOCIATE MEMBERS (*Continued*)

| | | Date of Membership. |
|---|---|---|
| GAYNOR, KEYES CHRISTOPHER. Cons. Engr., 405 Frances Bldg., Sioux City, Iowa | Jun.<br>Assoc. M. | June 30, 1910<br>Mar. 13, 1917 |
| GOULD, ROBERT RUTLEDGE. Asst. Engr., New York Municipal Ry. Corporation, 533 Tinton Ave., New York City | | April 17, 1917 |
| HAAS, EXUM MOSES. Sales Mgr., The International Steel Tie Co., 16702 Waterloo Rd., Cleveland, Ohio | | Mar. 13, 1917 |
| HARVELL, JOHN HERBERT. 800 West Redgate Ave., Norfolk, Va | | Jan. 15, 1917 |
| HEIMBUECHER, WALTER ALEXANDER. City Engr., City Hall, University City, Mo | | April 17, 1917 |
| HINDS, ARTHUR KLOCK. Engr., G. F. Hardy, 309 Broadway, Room 608, New York City | Jun.<br>Assoc. M. | Nov. 12, 1913<br>April 17, 1917 |
| HOLLINGSWORTH, HORACE WRIGHT. Vice-Pres., C. M. Pritchard Co., Inc., 910 North Elwood, Tulsa, Okla | | Jan. 15, 1917 |
| HOOPES, EDGAR MALIN, JR. Chf. Engr., City of Wilmington, 1303 Rodney St., Wilmington, Del | | April 17, 1917 |
| JAMISON, GEORGE HOLT. 306 O. & W. Station, Seattle, Wash | | April 17, 1917 |
| KNOETTGE, CARL HARMAN. Instr., Coll. of Eng., Cornell Univ., 109 De Witt Pl., Ithaca, N. Y | Jun.<br>Assoc. M. | Sept. 2, 1914<br>Mar. 13, 1917 |
| LABSAP, ALFRED HARRY. Asst., U. S. Engr. Office, P. O. Box 421, Vicksburg, Miss | | Mar. 13, 1917 |
| LUCCHETTI-OTERO, ANTONIO SEBASTIAN. Asst. Engr., Bureau of Public Works, Dept. of the Interior, San Juan, Porto Rico | Jun.<br>Assoc. M. | Jan. 3, 1911<br>Mar. 13, 1917 |
| OBERMEYER, WALTER SCOTT. Asst. Engr., J. Toner Barr, 934 St. James St., Pittsburgh, Pa | Jun.<br>Assoc. M. | April 7, 1915<br>April 17, 1917 |
| PEABODY, ALONZO ORBAN. Lewiston, N. Y | | April 17, 1917 |
| PHARIS, LE ROY MASTERS. Asst. Engr., Utah Power & Light Co., Salt Lake City, Utah | | April 17, 1917 |
| RADER, FRANKLIN KEARNS. (Rader Bros.), Lewisburg, W. Va | | April 17, 1917 |
| SHIELDS, JAMES RALPH. Engr. of Tests, Testing Laboratory, Univ. of California, 2119 McKinley Ave., Berkeley, Cal | Jun.<br>Assoc. M. | Oct. 1, 1912<br>Mar. 13, 1917 |
| SMALLMAN, RALPH ALCORN. Engr. and Contr. (Smallman-Brice Co.), 1617 Am. Trust Bldg., Birmingham, Ala | Jun.<br>Assoc. M. | Jan. 31, 1911<br>Mar. 13, 1917 |
| STANDISH, SEYMOUR. Chf. Engr., Standish & Allan, 820 Consumers Bldg., Chicago, Ill | | April 17, 1917 |
| TATE, ROBERT L'HOMMEDIEU. 134 Herkimer St., Buffalo, N. Y | Jun.<br>Assoc. M. | Mar. 4, 1913<br>April 17, 1917 |

ASSOCIATE MEMBERS (*Continued*) Date of Membership.

TAYLOR, HAROLD ALEXANDER. Supt. of Constr., Prack & Perrine, 7239 Penn Ave., Pittsburgh, Pa ........ April 17, 1917

WALLACE, DAVID ALEXANDER. With Imperial Munitions Board, 108 Gloucester St., Ottawa, Ont., Canada ..... Mar. 13, 1917

JUNIORS

BLY, EDWIN PRESCOTT. Draftsman and Engr., Standard Oil Co., Olympic Club, San Francisco, Cal ......... April 17, 1917

BROWN, HORATIO WHITTEMORE. Elm St., Concord, Mass. . April 17, 1917

COX, JOEL BEAN. County Engr., County of Maui, Wailuku, Maui, Hawaii ........................ Mar. 13, 1917

DENHAM, DONALD POWER. Care, Leonard Constr. Co., Box 36, Packer's Station, Kansas City, Mo ........... April 17, 1917

GOMEZ, ERNESTO. 2106 Sedgwick St., Chicago, Ill ......... Mar. 13, 1917

KING, HOWARD LANGDON. Junior Asst., Public Service Comm., 559 West 164th St., New York City ......... April 17, 1917

LOVE, JOSEPH EUGENE. 5648 Ridge Ave., Chicago, Ill ...... Nov. 28, 1916

MUENSTER, ROLAND AUGUST. Asst. City Engr., 1211 Rio Grande St., Austin, Tex ..................... Mar. 13, 1917

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## CHANGES OF ADDRESS

MEMBERS

BEELER, JOHN ALLEN. 301 West 108th St., New York City.

BILLIN, CHARLES EMERY. Bedford Hills, N. Y.

BLAIR, MCCREA PARKER. 17 Dundum Pl., Winnipeg, Man., Canada.

CHARLES, LA VERN JOHN. Asst. State Highway Engr., Santa Fé, N. Mex.

CHENEY, SHERWOOD ALFRED. Maj., Corps of Engrs., U. S. A., Fort Sam Houston, Tex.

CLAYTON, ROBERT MORRIS. Cons. Engr., Route 4, Asheville, N. C.

CLAYTON, THOMAS WILEY. With Chicago Elev. Rys., 322 Peoria Ave., Dixon, Ill.

COLLINS, EMMETT FILMORE. Asst. Engr., St. L. & S. F. R. R., 631 Frisco Bldg., St. Louis, Mo.

DEVIN, GEORGE. Room 626, Frisco Bldg., St. Louis, Mo.

DURHAM, EDWARD MIALL, JR. Care, So. Ry., 1300 Pennsylvania Ave., Washington, D. C.

FLAGLER, CLEMENT ALEXANDER FINLEY. Lt.-Col., Corps of Engrs., U. S. A., Army War Coll., Washington, D. C.

FRANCIS, HENRY NEWTON. 1526 Broad St., Providence, R. I.

FRENCH, MANSFIELD JOSEPH. Civ. Engr. and Archt., 707 Sumner Ave., Syracuse, N. Y.

FULLER, WILLIAM BARNARD. Cons. Engr., P. O. Box 97, Pierreville, Que., Canada.

GOODRICH, ERNEST PAYSON. Cons. Engr., 261 Broadway, New York City.

MEMBERS (*Continued*)

GREGORY, JOHN HERBERT. Cons. Hydr. Engr. and San. Expert, 170 Broadway, New York City.

HALL, LOUIS WELLS. Hydr. Engr., Aluminum Co. of America, Oliver Bldg., Pittsburgh, Pa.

HERING, RUDOLPH. Cons. Engr., 170 Broadway, New York City.

HORROCKS, JOHN IRVIN. (B. H. M. Lumber Co.), Tolt, Wash.

HOXIE, RICHARD LEVERIDGE. Brig.-Gen., U. S. A. (*Retired*), Iowa City, Iowa.

JOHNSON, CHARLES. Asst. Engr., Coventry Land & Impvt. Co., 403 Ohio Boulevard, Akron (Res., 62 South Towa Ave., Kenmore), Ohio.

JONES, ARTHUR LEWIS. Engr., Glendale & Montrose Ry., Care, H. Hawgood, 722 E. W. Hellman Bldg., Los Angeles, Cal.

KERSTING, FELIX JOHN. (Pope Constr. Co.), P. O. Box 533, Jefferson City, Mo.

LAHMER, JOHN ALOYSIUS. Drainage Engr., Mo. Pac. Ry., 1055 Railway Exchange, St. Louis, Mo.

LAYFIELD, ELWOOD NORMAN. 61 Broadway, Room 2216, New York City.

LEWIS, SIDNEY FRANCIS. Res. Engr., S. E. Arkansas Levee Dist., Arkansas City, Ark.

MCCONNELL, JOHN LORENZO. Asst. Archt., Central Mfg. Dist., 1118 West 35th St., Chicago, Ill.

MASON, FRANCIS. (Cadmus Constr. Co.), 120 Liberty St., New York City.

MATTIS, GEORGE. Asst. Div. Engr., California Highway Comm., Div. IV, Rialto Bldg., San Francisco, Cal.

NAYLOR, EZRA BOOTH. 201 Fremont St., Peekskill, N. Y.

O'BRIEN, ARTHUR. County Engr., 1627 Sunset Ave., Utica, N. Y.

OSBORN, FRANK CHITTENDEN. Cons. Engr., Osborn Eng. Co., 2848 Prospect Ave., Cleveland, Ohio.

PARDEE, JAMES THOMAS. Cons. Engr., P. O. Box 226, Midland, Mich.

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| | Date of Reinstatement. |
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| BUSHNELL, HOWARD EMORY | April 17, 1917 |
| DODD, JOHN HUGH | April 17, 1917 |

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MEMBERS

| | Date of Resignation. |
|---|---|
| BROHM, WILLIAM CARL | April 17, 1917 |

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| BURNHAM, FREDERIC WATERMAN | April 17, 1917 |
| JUSTICE, GED HARDY | April 17, 1917 |
| WINCHESTER, PHILIP HAROLD | April 17, 1917 |

JUNIORS

| | |
|---|---|
| DAVIS, WILLIAM EILERT | April 17, 1917 |
| JONES, CHARLES HYLAND | April 17, 1917 |
| WARFEL, ADAM COOPER | April 17, 1917 |
| WELLS, CHESTER GORDON | April 17, 1917 |

## DEATHS

BEARDSLEY, ARTHUR. Elected Associate, September 1st, 1875; Member, September 2d, 1891; date of death unknown.

BERGEN, VAN BRUNT. Elected Member, June 17th, 1868; died April 27th, 1917.

DAVIS, HAROLD. Elected Associate Member, October 2d, 1901; died March 25th, 1917.

HYDE, WILLIAM HERBERT. Elected Junior, April 30th, 1901; Associate Member, June 4th, 1902; died April 15th, 1917.

VEUVE, ERLE LEROY. Elected Junior, September 3d, 1901; Member, February 2d, 1909; died March 25th, 1917.

WRIGHT, EDWARD THOMAS. Elected Member, February 3d, 1886; died March 29th, 1917.

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### (April 1st to April 30th, 1917)

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#### LIST OF PUBLICATIONS

*In the subjoined list of articles, references are given by the number prefixed to each journal in this list:*

(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Indicator*, Hoboken, N. J., 50c.
(9) *Industrial Management*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record*, New York City, 15c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Engineering Journal*, New York City, 10c.
(25) *Railway Mechanical Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Coal Age*, New York City, 10c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.

(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Drop Forger,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Gas Journal,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
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(76) *Brick,* Chicago, Ill., 20c.
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(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
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(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
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(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
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(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
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(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
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General Layout of an Ideal Through a Power Plant.* Clarence A. [illegible]. (70) Feb. [illegible].
The Wisconsin River Hydraulic [illegible]. Ralph Wilson. (44) [illegible].
Cement and Insulating Installation. [illegible]. Robert L. Addy. (72) Feb. 7.
Mechanical Effects of Lateral Bending in Three Elements.* [illegible] Instruments. (72) Feb. 1.
Distribution Pressure in the Vertical Boiler.* Investigations of Messrs. A. E. Tayler and H. J. Kennedy. (44) Serial beginning Feb. 2.
Some Practical Properties of the Thin Plate Affecting Valuation of [illegible]. W. R. Chipperfield. (60) Feb. 6.
Milling the Margin Once.* Joseph A. Stanley. (72) Feb. 8.
Machining Operations on a Four-Cylinder Automobile Engine.* Robert Mawson. (72) Feb. 8.
Filter [illegible].* A. E. Grayson Wilson. (70) Feb. 13.
Casting Molding Plates and Handling Their Flasks.* Clarence A. [illegible]. (70) Feb. 13.
Making Dies for Typewriter Type.* Frank A. Stanley. (72) Feb. 15.
Electric-Driven Housing-Type Planers.* (72) Feb. 15.
Oil-Washing for Engines and Tailstocks. W. G. Adams. (60) Feb. 20.
The Possible Partial Substitution of Retort Coke for Anthracite Coal in the Manufacture of Sulphate of Ammonia.* C. Parrish. (60) Feb. 20.
A New Method of Extracting the Explosive Constituents from Coal Gas.* M. Lessing. (60) Feb. 20.
New Methods of Measuring Gas Under Varying Conditions of Temperature and Pressure.* C. M. Walter and H. D. Marsh. (Paper read before the Midland Junior Gas Assoc.) (60) Feb. 20.
Long Endurance of Wrought-Iron Chain Cable at Boston Navy Yard. F. G. Coburn. (72) Feb. 22.
Finding Stable Dimensions for Drawing Cups.* P. Rabine. (42) Mar.
Shifting and Sorting for Machine Shops. Joseph Horner. (42) Mar.
Broaching Typewriter Parts.* Frank A. Stanley. (72) Mar. 1.
Novel Methods Employed in Cutting Tubes from Saltwater Condensers. Richard [illegible] Jr. (24) Mar. 2.
Bending, Piercing and Forming Tools for a Typewriter Part.* Frank A. Stanley. (72) Mar. 7.
Heat-Treating Plant of the Home Pressure Gear Corporation.* E. A. Suverkrop. (72) Serial beginning Mar. 7.
Using the Electric Welder on Parts for Automobiles.* Robert Mawson. (72) Mar. 7, Apr. 4.
Introduction of By-Product Coke into Gas Supply, Without Loss of Available Coals for Coking.* W. H. Hancock. (24) Mar. 9.
Methods Followed in the Manufacture of Hydroplanes.* Robert Mawson. (72) Mar. 14.
Steam Turbines for Land Purposes.* W. B. [illegible]. (Paper read before the Manchester Assoc. of Engrs.) (11) Serial beginning Mar. 15.
Lubrication of Steam Turbines.* T. C. Thomsen. (12) Serial beginning Mar. 16.
Twist Drill Grooving Machine.* (11) Mar. 16.
Gas. Reginald Crewe. (Paper read before the Scottish Junior Gas Assoc., Western District.) (60) Mar. 20.
Standardization of Airplane Parts. F. G. Diffin. (72) Mar. 22.
Turbine Wheel Fits and Journals. Pennsylvania R. R. Practice.* (72) Mar. 22.
Precision Grinding. (From paper read before Manchester Assoc. of Engrs.) (12) March 23.
Notes on Gas-Works Practice Under War Conditions. George H. Jones. (Paper read before the Midland Junior Gas Assoc.) (60) Mar. 27.
Gearing, their Design and Manufacture.* W. G. Groocock. (72) Mar. 29.
Special Attachments for Making Typewriter Parts.* (72) Mar. 29.
Aerial Wire Ropeways.* J. W. White. (Abstract of paper read before the South Wales Inst. of Engrs.) (57) Mar. 29.
The Recovery of Benzol from Gas.* F. W. Sperr. (57) Mar. 30.
Practical Notes on Screw Threads.* W. G. Groocock. (11) Mar. 30.
Would Make the Selection of Domestic Heaters for Sale by Gas Company, Essentially an Engineering Proposition.* W. C. Bastian. (From paper read before the New England Gas Light & Coke Co. of Chicago.) (24) Mar. 31.
New Furnace, it is Claimed, Enables Combustion to be Completely Localized.* (24) Mar. 31.
Forming Tools for Shaping Radiator Rods.* P. Rabine. (42) Apr.
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Pulverized Coal Plant for the Santa Fe.* (25) Apr.
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The Pencil Electrode Method of Welding for Boiler Joints.* E. A. Wildt. (55) Apr.
Probable Requirements in Machine Tools.* A. M. Sosa. (55) Apr.
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Economy in Hacksaw Blades. Frank E. Merriam. (72) Apr. 5.
Producing British Screw Gages.* I. W. Chubb. (72) Apr. 5.
Machining an Aeroplane Motor Crank Case.* Fred F. Bowen. (72) Apr. 5.
The Use of Zirconia as a Refractory Material. J. A. Audley. (Abstract of paper read before the Ceramic Soc.) (11) Apr. 6.
A Caterpillar Tractor.* (12) Apr. 6.
Preventing Air-Driven Pumps from Freezing.* James Humes. (16) Apr. 7.
Recent Developments in the Design of Hoisting Engines.* Frederick W. O'Neil. (16) Apr. 7.
Uses and Costs of Aërial Tramways.* William C. Kuhn. (16) Apr. 7.
Welding by the Electric Arc Process.* (18) Apr. 7.
Use of Mine Water as Boiler Feed. Edwin M. Chance. (45) Apr. 7.
Gas for Heating Galvanizing Baths Meets Coke Fuel Cost.* William J. Harris, Jr. (24) Apr. 7.
Construction and Operation of a 36-Ft. Round Face Brick Kiln.* T. N. McVay. (76) Apr. 10.
Heat Balance of an Absorption Plant.* Julian C. Smallwood. (64) Apr. 10.
Energy Stored in a Boiler Under Pressure.* F. R. Low. (64) Apr. 10.
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Screw Thread Measurement.* (Abstract of paper read before the Liverpool Eng. Soc.) (20) Apr. 12.
Efficiency of Waste-Heat Boilers.* H. A. Reichenbach. (20) Apr. 12.
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Methods for Handling Coal and Ashes; Typical Equipments are Those in the Power Plants of the Norfolk & Western Railway, the Springfield (Mass.) Street Railway and the Buffalo General Electric Company.* (17) Apr. 14.
Laying a Submarine Gas Main in Winter.* A. F. Davy. (Paper read before the Wisconsin Gas Assoc.) (83) Apr. 16.
Natural-Gas Pumping Station at Roystone.* Harold B. Bernard. (64) Apr. 17.
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Design of Keyway Broaches.* Walter G. Groocock. (72) Apr. 19.
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Suggestions Relative to the Manufacture of Parts to Limit.* F. H. Bogart. (72) Apr. 19.
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Expediting Transfer of Work in Forge Shops.* Charles C. [illegible]. (62) Apr.
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The Design of [illegible] Boiler [illegible].* [illegible]. (72) Apr. 5.
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The Use of [illegible] as a [illegible] Material. [illegible]. (Abstract of paper read before the [illegible].) (11) Apr. 6.
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Recent Developments in the Design of Hoisting Engines.* [illegible] W. [illegible]. (16) Apr. 7.
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Time Studies on Automatic Machines.* [illegible]. (72) Apr. 12.
An Unusual Metal Pattern for a [illegible].* [illegible]. (72) Apr. 12.
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Laying a Submarine Gas Main for [illegible].* [illegible]. (Paper read before the Wisconsin Gas Assoc.) (62) Apr. 16.
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Design of Keyway Broaches.* [illegible]. (72) Apr. 19.
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Machine Operations in the [illegible] Shop.* (72) Apr. 19.
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Blast Furnace and Steel Mill Power Plants.* Richard H. Rice and Sanford A. Moss. (58) Mar.
Effect of Time Factor in Heating Steels.* Carle R. Hayward and S. S. Raymond. (62) Mar.
Cost for Fuel to Smelt and Assay Over a Million Dollars Worth of Gold and Silver Ore, Using Gas, is $225.* C. E. Gray. (24) Mar. 3.
Blast Furnace the Ideal Appliance for Making Hard Candies.* Gilbert C. Shadwell. (24) Mar. 10.
Economy of Gas as Carburizing Agent in Case Hardening of Small Pieces Has Been Demonstrated. (24) Mar. 10.
Rate and Ultimate Temperature of Heating and Methods of Cooling Important Operations in Steel Hardening. W. A. Ehlers. (From *Bulletin* of the National Commercial Gas Assoc.) (24) Mar. 17.
Tube-Mill Practice in Rhodesia.* A. W. Allen. (16) Mar. 17.
On an Electric Resistance Furnace for Melting in Crucibles.* H. C. Greenwood and R. S. Hutton. (Paper read before the Institute of Metals.) (11) Mar. 23.
Metal Melting in a Simple Crude-Oil Furnace.* H. S. Primrose. (Paper read before the Institute of Metals.) (11) Mar. 23.
Analysis of Aluminum Dust. J. E. Clennell. (16) Mar. 24.
Metal Melting as Practised at the Royal Mint.* W. J. Hocking. (Paper read before the Inst. of Metals.) (11) Mar. 30.
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A Comparison System for Determining and Standardizing the Grain Size of Annealed Brass.* Geo. A. Miller, Jr. (105) Apr. 1.
The Cementation of Iron and Steel.* Ernest Edgar Thum. (105) Apr. 1.
Phosphorus in Steel: Its Determination by a New Method Using an Ammoniacal Water Solution of Molybdate. Charles M. Johnson. (20) Apr. 5.
Brass Manufacture in a Modern Plant.* W. F. Freeland. (20) Apr. 5.
Surface Combustion Used in Galvanizing.* William J. Harris, Jr. (20) Apr. 5.
New Design Bar Mill at Youngstown.* (20) Apr. 5.
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Metal Melting by Means of High-Pressure Gas.* C. M. Walter. (Paper read before the Inst. of Metals.) (11) Apr. 6.
Sulphur Di-Oxide as a Flotative Agent.* (103) Apr. 7.
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American Steel Slags and Their Uses. Edwin F. Cone. (20) Apr. 12.
Influence of Gases in Cast Metal.* J. E. Fletcher. (Abstract of paper read before the Birmingham Branch of the British Foundrymen's Assoc.) (47) Apr. 13.
Aluminum-Dust Precipitation Tests. P. H. Crawford. (103) Apr. 14.
Tempering of Steel by Oil Bath Insures Uniform Heating with no Chance to Overheat. (24) Apr. 14.
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The [illegible] of Flotation Processes.* Robert S. Lewis. (105) Apr. 7.
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Influence of [illegible] in Cast Metal.* J. E. Fletcher. (Abstract of paper read before the Birmingham Branch of the British Foundrymen's Assoc.) (47) Apr. 13.
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Tempering of Steel by Oil Bath Insures Uniform Heating with no Chance [illegible] (24) Apr. 14.
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Bearing Bronzes and the Microscope.* C. H. Bierbaum and Verne Skillman. (20) Apr. 19.
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New East Rand Mines. A. Cooper Key. (16) Mar. 17.
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Refrigeration and Its Application to Mining. Andrew D. Brydon. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Mar. 23.
Comparison of Electric Winding Systems. S. E. T. Ewing. (Abstract of paper read before the South African Inst. of Elec. Engrs.) (57) Mar. 23.
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Mining Laws of Brazil. Henry Thomas. (16) Mar. 24.
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Winter Grinding and Screening: Ideas on a Subject of Wide Interest, Gained Where the Frost King Labors Long and Hard.* Ernest W. Knapp. (76) Mar. 27.
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Nevada's Latest Gold District.* Fred J. Siebert. (103) Mar. 31.
Tin and Tungsten in South Dakota.* John Bland. (103) Mar. 31.
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Unwatering an Inclined Shaft.* James E. Harding. (16) Mar. 31.
The Lordsburg District, New Mexico.* Faris V. Bush. (16) Mar. 31.
Nitro-Starch as an Explosive.* S. S. Sadtler. (105) Apr. 1.
Electrification of a Group of Collieries.* Harold Green. (22) Apr. 6.
The Raising of Coal from Vertical Shafts. Hugh Bramwell. (22) Apr. 6.
Losses Incident to the Caving of Orebodies.* R. G. Sampson. (103) Apr. 7.
Development in Safety Lamps.* James Ashworth. (45) Apr. 7.
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Successful Electrical Installations.* C. L. Packard. (45) Apr. 7.
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Loading Pocket in Burra Burra Mine.* Walter R. Hodge. (16) Apr. 7.
The Testing and Application of Hanmer Drills.* Benjamin F. Tillson. (Paper read before the Am. Institute of Min. Engrs.) (16) Apr. 7.
Underground Traming.* David B. Scott. (16) Apr. 7.
Underground Churn Drilling at Old Dominion.* Guy N. Bjorge. (16) Apr. 7.
Mine-Pumping Equipment.* Whitman Symmes. (16) Apr. 7.
Mining Methods Illustrated in Miniature.* W. R. Crane. (16) Apr. 7.
Recording of Accidents, and Safety Measures Employed by the New Jersey Zinc Co.* Benjamin F. Tillson. (16) Apr. 7.
Methods of Underground Metal Mining.* F. W. Sperr. (16) Apr. 7.
The Coronado Top-Slicing Method.* Peter B. Scotland. (16) Apr. 7.
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[illegible] and the [illegible]* [illegible] (50) Apr. 13.
New Electric Steel Plant and Rolling Mill. (20) Apr. 26.
An Improved System of [illegible]* [illegible] (101) Apr. 26.

**Military.**

New Features of Hydraulic Shock [illegible]* [illegible] (72) Mar. 9.
[illegible] Appointments in the [illegible] (14) Mar.
United States [illegible] (11) Mar.
The [illegible]* [illegible] (11) Mar. 16.
United States [illegible] (72) Mar. 23.
[illegible] Armored [illegible] (40) Apr. 6.
Ammunition for [illegible]* [illegible] (11) Apr. 13.
The [illegible]* (72) Apr. 20.
[illegible] (11) Apr. 20.
A Method of [illegible] (11) Apr. 20.
The "Anti-[illegible]"* [illegible] (41) Apr. 26.
Organization and Training of the New Armies. (11) Mar. [illegible]
[illegible] (11) Mar. 30.

**Mining.**

What [illegible] Congress [illegible] (22) Mar. 10.
New [illegible] (16) Mar. 17.
Preliminary Report of the Mineral Production of Canada for 1916. (16) Mar. 17.
New Work of the Bureau of Mines.* (16) Mar. 17.
[illegible] and its Application to Mining. [illegible] (Paper read before the National [illegible] of [illegible]) (22) Mar. 23.
[illegible] (Abstract of paper read before the [illegible]) (57) Mar. 23.
[illegible] and Cooling [illegible]* [illegible] (57) Mar. 23.
[illegible] of the [illegible] (45) Mar. 23.
The [illegible] Mine [illegible]* [illegible] (16) Mar. 24.
Mining [illegible] [illegible] (16) Mar. 24.
[illegible] of Lead Ore.* [illegible] (16) Mar. 24.
[illegible] and [illegible] (Paper read before [illegible] Lodge and [illegible]* [illegible] (73) Mar. 23.
Hydraulic [illegible] Plant at the [illegible] (57) Mar. 30.
[illegible]* [illegible] (16) Mar. 31.
The [illegible] in South [illegible]* [illegible] (107) Mar. 31.
[illegible] an Electric [illegible]* [illegible] (16) Mar. 31.
[illegible] an Inclined Shaft.* [illegible] (16) Mar. 31.
The [illegible] District, New Mexico.* [illegible] (16) Mar. 31.
[illegible] of an [illegible]* [illegible] (104) Apr. 1.
[illegible] of a Group of Collieries.* [illegible] (22) Apr. 6.
The Raising of Coal from Vertical Shafts. [illegible] (22) Apr. 6.
[illegible] Incident to the Caving of [illegible]* [illegible] (104) Apr. 6.
Development in Safety Lamps.* [illegible] (45) Apr. 7.
The Design of Headframes.* [illegible] (16) Apr. 7.
[illegible] Electrical Installations.* [illegible] (45) Apr. 7.
The United Verde Underground [illegible]* [illegible] (16) Apr. 7.
Loading Pockets in [illegible] Mine.* [illegible] (16) Apr. 7.
The Testing and Application of [illegible]* [illegible] (Paper read before the [illegible] of Mine [illegible]) (16) Apr. 7.
Underground Tramways.* [illegible] (16) Apr. 7.
Underground [illegible] Drilling at [illegible]* [illegible] (16) Apr. 7.
Mine-Pumping Equipment.* [illegible] (16) Apr. 7.
Mining Methods Illustrated in Miniature.* W. H. Crane. (16) Apr. 7.
Recording of Accidents and Safety Measures Employed by the New Jersey Zinc Co.* Benjamin F. Tillson. (16) Apr. 7.
Methods of Underground Metal Mining.* [illegible] (16) Apr. 7.
The [illegible] Top-Slicing Method.* [illegible] (16) Apr. 7.
Winding [illegible] in Mines. (27) Apr. 7.
An Improved Coal Drill.* (45) Apr. 7.

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New Gathering Pump.* (45) Apr. 7.
Luck and Accuracy in Borehole Location.* L. Netland. (45) Apr. 7.
Large Electric Hoist.* (45) Apr. 7.
Blue Diamond Coal Co.'s Plant at Cardiff, Ky.* H. Reisser. (45) Apr. 7.
A Successful Gasoline Locomotive.* (45) Apr. 7.
Ramsay Mine-Run Sampler.* H. S. Grismer. (45) Apr. 7.
Successful Colliery Management.* H. M. Crankshaw. (45) Apr. 7.
Reconstruction of the Benham Tipple.* H. B. Cooley. (45) Apr. 14.
Yampa Coal Field in Colorado.* F. W. Whiteside. (45) Apr. 14.
Ventilation in the Rincon Mine.* Blamey Stevens. (103) Apr. 14.
Gold Dredging in Tierra del Fuego.* Lyman Chatfield. (16) Apr. 14.
Methods of Mining Big Orebodies.* Robert A. Kinzie. (103) Apr. 21.
A Narrow-Gage Alternating Current Mine Locomotive.* Rudolph W. Van Norden. (16) Apr. 21.
Clay County Graphite District of Alabama.* Irving Herr. (16) Apr. 21.
Economics of Grade Revision for Underground Haulage.* R. D. Brown. (45) Apr. 21.
Longwall Mining at Steubenville, Ohio.* Wilbur G. Burroughs. (45) Apr. 21.
Grade Revision of Underground Roads.* R. D. Brown. (45) Apr. 28.
Le Gisement de Mineral der Fer Lorrain; Etude Minière et Métallurgique.* J. Tribot-Laspière. (33) Apr. 7.

**Miscellaneous.**

Correction of Thermometers in the Determination of Boiling Points.* Kenneth Norton. (66) Jan. 16.
Time Studies for Rate Setting as Originated by Dr. F. W. Taylor. Carl G. Barth. (72) Feb. 1.
Object and Method of Taking Time Studies for Rate Setting.* Dwight V. Merrick. (72) Feb. 8.
Production Time Studies to Check Rates and Operations.* Dwight V. Merrick. (72) Feb. 15.
Relation of Public Utilities to the Public. W. W. Freeman. (4) Mar.
Industrial Democracy with Particular Reference to the Relation between Capital and Labor. George Weston. (4) Mar.
Organizing a Time-Study Department.* Dwight V. Merrick. (72) Mar. 8.
Fertilisers and Their Supply in War Time. John Augustus Voelcker. (29) Mar. 16.
Office Routine and the Planning System.* H. C. Brumbaugh. (17) Mar. 17.
Regulating Materials and Supplies.* B. J. Yungbluth. (17) Mar. 17.
Technical School Buildings: Their Accommodation and Equipment. (104) Mar. 23.
Cultivation of Lands Order, 1916. Reginald Brown. (104) Mar. 30.
The Organic Nitrogen Compounds of Soils and Fertilizers. Elbert C. Lathrop. (3) Apr.
Graphical Control on the Exception Principle for Executives.* Frank B. Gilbreth. (55) Apr.
The Schmidt Market Ammonia Explosion. George P. Carroll. (64) Apr. 3.
A Handy Chart for Weighing Excavated Materials.* (96) Apr. 12.
Engineering Methods Must Replace Paternalism in the Handling of Labor. Frances A. Kellor. (13) Apr. 12.
The Fuels of Texas. A. C. Scott. (45) Apr. 14.
Central Control of Production Methods. Francis J. G. Reuter. (72) Apr. 19.
American Sources of Nitrogen.* Thomas H. Norton. (46) Apr. 21.
The Problem of the Crippled Soldier. How to Put Him on the Pay Roll.* Frank B. Gilbreth and Lillian M. Gilbreth. (19) Apr. 28.
Mechanical Aids in Accounting.* A. G. Schwenke. (17) Apr. 28.
Benzene, Toluene and Xylene from Petroleum. Gustav Egloff. (105) May 1.
Synthetic Rubber.* Andrew H. King. (105) May 1.
Effusion Method of Determining Gas Density.* Junius David Edwards. (105) May 1.
La Rééducation Professionnelle des Mutilés de Guerre.* Amar. (92) Jan.
Le Porte-Outils Jullien Destiné aux Mutilés d'un Bras.* Max Ringelmann. (92) Jan.

**Municipal.**

Influencing Suburban Communities to Adopt Gas for Street Lighting by Dwelling Upon Its Convenience for Domestic Purposes.* (24) Mar. 3.
Street Cleansing Methods: Halifax Corporation and the Question of Control. (104) Mar. 16.
The Control of Street Architecture. (104) Mar. 16.
Lay 6-In. Gas Main Across Frozen River at Total Cost of Two Hundred Forty-seven and One-half Dollars.* (Abstract of paper read at Annual Meeting of Wisconsin Gas Assoc.) (24) Mar. 24.
Poor Lighting Promotes Public Ownership. C. E. Clewell. (27) Apr. 7.

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**Mining—Continued.**

New Centrifugal Pump.* (18) Apr. 4.
[illegible] and Accidents in [illegible] Locations.* [illegible]. (43) Apr. 7.
Large Electric Hoists.* (45) Apr. 7.
Blue Diamond Coal Co.'s Plant at Hazard, Ky.* H. [illegible]. (47) Apr. 7.
A Monorail Gasoline Locomotive.* (45) Apr. 7.
Summer Ventilation Problems.* R. B. [illegible]. (45) Apr. 7.
Successful Colliery Management.* H. M. Chamberlain. (45) Apr. 7.
Reconstruction of the Dawson Tipple.* H. H. Conley. (45) Apr. 14.
Tonopah Gold Field in Operation.* F. W. McKenzie. (45) Apr. 14.
Ventilation in the Simon Mine.* Edward Brewer. (103) Apr. 14.
Gold Dredging in Tierra del Fuego.* Carmen Chatfield. (103) Apr. 14.
Methods of Mining Big Orebodies.* Robert A. Kinzie. (103) Apr. 21.
A Narrow-Gauge Automatic Vertical Steam Locomotive.* Gardner W. Van Dervoort. (10) Apr. 21.
Clay County Graphite Mines in Alabama.* Irving Herr. (16) Apr. 21.
Examples of Grade Mechanics and Underground Haulage.* G. B. Brown. (45) Apr. 21.
Longwall Mining at Shamokin, Wash.* Walter H. [illegible]. (45) Apr. 21.
Steam Heating of Underground Shafts.* R. B. Brown. (45) Apr. 28.
Le Classement du Minerai dans les Lorraine. Étude Minière et Métallurgique.* J. Tribot-Laspière. (54) Apr. 6.

**Miscellaneous.**

Correction of Thermometers in the Determination of Boiling Points.* Nicholas Norton. (46) Jan. 26.
Time Studies for Rate Setting as Developed at the U. S. Arsenal. Dwight V. Merrick. (72) Feb. 1.
Observation Method of Taking Time Studies for Rate Setting.* Dwight V. Merrick. (72) Feb. 8.
Production Time Studies in Check Hours and Operations.* Dwight V. Merrick. (72) Feb. 15.
Relation of Public Utilities to the Public. W. W. Freeman. (4) Mar.
Industrial Democracy with Particular Reference to the Relation between Capital and Labor. George Weston. (4) Mar.
Organizing a Time-Study Department.* Dwight V. Merrick. (72) Mar. 8.
Fertilizers and Their Supply in War Time. John Augustus Voelcker. (29) Mar. 15.
Office Routine and the Planning System.* H. C. Urquhart. (17) Mar. 17.
Requisitioning Materials and Supplies.* H. J. Vanderhoff. (17) Mar. 17.
Technical School Buildings: Their Accommodation and Equipment. (104) Mar. 23.
Cultivation of Lands Order, 1918. Reginald Brooks. (104) Mar. 29.
The Organic Nitrogen Compounds of Soils and Fertilizers. Elbert C. Lathrop. (4) Apr.
Graphical Control on the Exception Principle for Executives.* Frank B. Gilbreth. (88) Apr.
The Schmidt Market Analysis Equipment. George P. Carroll. (64) Apr. 4.
A Handy Chart for Weighing Excavated Materials.* (96) Apr. 11.
Machine-like Methods Must Replace Guesswork in the Handling of Labor. Frances A. Kellor. (13) Apr. 13.
The Units of Force. A. G. Webster. (48) Apr. 14.
Central Control of Production Methods. Francis J. G. Keesler. (72) Apr. 18.
American Sources of Nitrogen.* Thomas H. Norton. (40) Apr. 21.
The Problem of the Crippled Soldier: How to Put Him on the Pay Roll.* Frank B. Gilbreth and Lillian M. Gilbreth. (19) Apr. 25.
Mechanical Aids to Accounting.* J. G. Schwenk. (17) Apr. 28.
Benzene, Toluene and Xylene from Petroleum. Gustav Egloff. (105) May 1.
Synthetic Rubber.* Andrew H. King. (105) May 1.
Efficient Method of Determining Gas Density.* Junius David Edwards. (105) May 1.
La Coordination Professionnelle des Métiers du Génie.* Anon. (92) Jan.
Le Porte-Outils Fulton, [illegible] d'un Moteur.* Max Ringelmann. (92) Jan.

**Municipal.**

Interesting Suburban Communities to Adopt Gas for Street Lighting by Dwelling Upon its Convenience for Domestic Purposes.* (24) Mar. 2.
Street Cleaning Methods: [illegible] Comparisons and the Question of Control. (104) Mar. 16.
The Control of Street Architecture. (104) Mar. 16.
Lay 6 in. Gas Main Across [illegible] River at Total Cost of Two Hundred Forty-seven and One-half Dollars.* (Abstract of paper read at Annual Meeting of Wisconsin Gas Assoc.) (24) Mar. 23.
Poor Lighting Promotes Public Ownership. C. E. Clewell. (27) Apr. 6.

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**Railroads.**

An Uptodate Wheel Shop.* Frank A. Stanley. (72) Jan. 4.
Wheel-Shop Dismounting Press.* Frank A. Stanley. (72) Jan. 11.
Axle Work in a Railroad Shop.* Frank A. Stanley. (72) Jan. 25.
Recent American Heavy Goods Locomotives.* (12) Mar. 16.
The Value of Standards to the Railway Industry. A. L. Broomall. (17) Mar. 17.
Motor Maintenance at Providence.* (17) Mar. 17.
Dump Cars Save $1 600 per Mile.* C. H. Clark. (17) Mar. 17.
Killing Weeds by Chemical Treatment.* (17) Mar. 24.
New Passenger Tank Engines for the North Staffordshire Railway.* (12) Mar. 30.
Handling Freight on a Southern Line.* T. G. Brabston. (17) Mar. 31.
Track and Roadway. W. F. Carr. (Abstract of paper read before the Ill. Elec. Ry. Assoc.) (17) Mar. 31.
What are Fair Rates of Return? Halford Erickson. (17) Mar. 31.
Fire Protection, Inspection and Signalling in the Southern Pacific's Snowshed District.* (18) Mar. 31.
Export Shipment of Heavy Electric Locomotives.* (18) Mar. 31.
Reinforcing Freight Car Draft Sills.* Lewis K. Sillcox. (Abstract of paper read before the Car Foreman's Assoc. of Chicago.) (25) Apr.
The Design of Movable Point Crossings.* H. F. Heyl. (87) Apr.
Perforating Ties before Treatment.* O. P. M. Goss. (87) Apr.
Training Maintenance of Way Men. F. W. Thomas. (87) Apr.
Recent Patents Relating to Railways.* (21) Apr.
Signalling Practice on the New South Wales Government Railways. (21) Apr.
Electrical Equipment Repair Shops: Description of the Facilities for the New Haven at Van Nest, N. Y., for Handling Electrical Rolling Stock.* (25) Apr.
Modern Railway Shop Lighting.* (25) Apr.
Pennsylvania Locomotive Tests.* Andrew C. Loudon. (25) Apr.
Spanish Passenger Locomotives.* (25) Apr.; (15) Apr. 13; (18) Apr. 7.
Pennsylvania 85-Ton Hopper Cars: An All-Steel Coal Car Having a Light Weight of 60 000 Lb. and 3 228 Cu. Ft. Gross Cubical Capacity. (25) Apr.
Electrification of the Norfolk & Western Railway Company. C. H. Quinn. (65) Apr.
Operating Results from the Trunk Line Electrification of the Chicago, Milwaukee & St. Paul Railway.* R. Beeuwkes. (Paper read before the New York R. R. Club.) (25) Apr.
Placing Screw Spikes with Pneumatic Drills.* (15) Apr. 6.
Electric Arc Welding in the Pennsylvania Railroad Shops.* Robert Mawson. (72) Apr. 12.
McClellon Water-Tube Firebox.* (15) Apr. 13.
Railways at the Service of the Government.* (15) Apr. 13.
The Grade Separation Problem at Syracuse.* (15) Apr. 13.
Weble Automatic Train Stop.* (15) Apr. 13.
New Carhouse and Station in Milwaukee.* (17) Apr. 14.
Two-Car Trains for Suburban-Interurban Service.* H. S. Reed. (17) Apr. 14.
Electric Locomotives more Costly to Maintain. (13) Apr. 19.
Transverse Fissures in Rails.* Geo. W. Dress. (20) Apr. 19.
West Penn Shops and Shop Methods.* Daniel Durie. (17) Apr. 21.
D. U. R. Begins Construction of New Freight Terminal.* (17) Apr. 21.
Truck Equalization.* S. A. Bullock. (17) Apr. 21.
Unit-Construction Canopies are Built on Curve.* G. E. Quick. (13) Apr. 26.
Common Labor Hard to Hold on Job of Track Elevation, Rock Island Finds. (13) Apr. 26.
Special Cars and Heavy Forms Used to Reline Tunnel Between Trains.* (13) Apr. 26.
Signal Lamps with Reflectors.* (15) Apr. 27.
Heavy Locomotives for the Lehigh Valley.* (15) Apr. 27.
An All-Ingot Test for Steel Rails. Geo. F. Hand. (15) Apr. 27.
Reducing Load *vs.* Raising Yield Point as Rail Corrugation Cure.* Henry M. Sayers. (17) Apr. 28.
Organization of the West Penn Railways.* (17) Apr. 28.
Report on Pere Marquette and Cincinnati, Hamilton & Dayton Railway Systems by the Interstate Commerce Commission.* (18) Apr. 28.
Reclamation in the Track and Mechanical Departments of the Chicago Great Western Ry.* (18) Apr. 23.
Les Chemins de Fer Chinois, le Programme du Gouvernement Chinois pour leur Développement.* A.-J.-H. Charignon. (32) Dec., 1916.
Les Tarifs des Chemins de Fer Français Nécessité de les Augmenter.* P. Mallet. (33) Mar. 24.
Der Wechselstrom-Transformator in der elektrischen Traktion.* W. Kummer. (107) Mar. 10.
Die Drahtseilbahn Treib-Seelisberg.* H. H. Peter. (107) Serial beginning Mar. 17.

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Maintenance of Overhead on the Westchester.* (17) Mar. 17.
Analysis of Track Maintenance Costs.* R. C. Cram. (17) Mar. 17.
Economic Aspects of Franchises. F. W. Doolittle. (17) Mar. 17.
Maintaining Substation Efficiency. D. E. Crouse. (17) Mar. 24.
Standardizing Car-Yard Layouts in Detroit.* (17) Mar. 24.
Making a Traffic Survey for Workers.* (17) Mar. 31.
Cincinnati's Rapid-Transit Ordinance.* (17) Apr. 7.
Changing the Fare Collection System Complete Over Night.* (17) Apr. 7.
Analyses of P. R. T. Proposal.* (17) Apr. 7.
Progress on the Boston Subway.* (18) Apr. 14.
Chicago Traffic, Value and Finances; Methods used by Traction and Subway Commission in Checking Travel for Unified System: Cost and Present Value of Elevated Lines; Summary of Financial Plan.* (17) Apr. 14.
Third Unit Added to Detroit United Shops.* (17) Apr. 14.
Car Overhand on Curves.* S. L. Foster. (17) Apr. 21.

**Roads and Pavements.**

Old Macadam of Little Value as a Base for Concrete Pavement.* D. A. Thomas. (13) Mar. 1.
Tractive Resistance on Various Road Surfaces.* (60) Apr.
The Treatment of Wood Paving Blocks.* C. H. Teesdale. (60) Apr.
The Construction of Asphalt Pavements.* (60) Apr.
Standard Pavements for Los Angeles, Cal. (86) Apr. 4.
Broken Stone and Concrete Foundations for Pavements.* (86) Apr. 4.
Notes on the Maintenance of Gravel and Macadam Roads. (86) Apr. 4.
Method and Cost of Grading and Constructing Sidewalks for a Tract Improvement.* E. E. Glass. (86) Apr. 4.
Methods and Cost of Screening and Washing Gravel for Country Road Construction. (86) Apr. 4.
Connecticut Method of Maintaining Concrete Roads. (86) Apr. 4.
Methods and Cost of Cheaply Scarifying and Resurfacing Macadam.* Halbert P. Gillette. (86) Apr. 4.
Best Practice in Concrete Road Construction. H. E. Breed. (Abstract of paper read before Am. Road Builder's Assoc.) (96) Apr. 5.
Best Practice in Wood Block Pavements. Ellis R. Dutton. (Paper read before Am. Road Builders' Assoc.) (96) Apr. 5.
Dust Preventatives and Bituminous Binders.* G. Cameron Parker. (Paper read before Third Annual Conference on Road Construction.) (96) Apr. 5.
Profit or Loss in Hauling Road-Building Material, a Discussion of the Relative Merits of Teams, Tractors, Motor Trucks and Industrial Railways. A. D. Thompson. (13) Apr. 12.
Transportation of Materials.* George Hogarth. (Paper read before Third Annual Conference on Road Construction.) (96) Apr. 12.
Procedure for Highway Lighting Districts. E. B. Walthall. (111) Apr. 15.
The Contract and Specification for Paving. Charles A. Mullen. (Paper read before the Dominion Good Roads Congress.) (96) Apr. 19.
Cement Concrete Roads and Pavements. T. Harry Jones. (Abstract of paper read before the Dominion Good Roads Congress.) (96) Apr. 19.
Bituminous Macadam Roads. Wm. B. Sohier. (Abstract of paper read before the Dominion Good Roads Congress.) (96) Apr. 19.
Tunnel Under Hudson Designed for Vehicular Traffic, Engineers Recommend that Tube between New York and New Jersey be Built by Floating Sections to Place and Sinking.* (13) Apr. 19.
Broken Old Concrete as Aggregate for New Paving Foundation. Clark R. Mandigo. (13) Apr. 19.
Monolithic Brick Pavement Tried in the Northwest. A. P. Denton. (13) Apr. 19.
Unique Method of Handling Hot Oil on Maintenance Work. H. M. Lukens. (13) Apr. 26.
Weekly Report Forms Simplify Highway Engineers' Work. M. W. Watson. (13) Apr. 26.
Road Construction Costs Lowered by Close Attention to Details.* Lester H. Morris. (13) Apr. 26.
How Much Water Should be Used in Road Concrete?* (13) Apr. 26.
Tests Show What Parts of City Streets are Used by Traffic.* H. T. Hutchins. (13) Apr. 26.
Correction of Alignments and Grades in Existing Highways. A. Fraser. (96) Apr. 26.
Road Oils and Tars. Arthur H. Blanchard. (Paper read before the Can. and Inter. Roads Cong.) (96) Apr. 26.
Road Foundations and Drains. R. Crawford-Muir. (Paper read before the Cong. on Road Construction.) (96) Apr. 26.
Les Travaux d'Ouverture de Routes et de Chemins en Algérie, Pendant la Guerre. Vielle. (43) Nov., 1916.

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Maintenance of Overhead on the Washington.* (17) Mar. 17.
Analysis of Track Maintenance Costs.* R. C. Cram. (17) Mar. 17.
Economic Aspects of Extensions. T. W. Doolittle. (17) Mar. 17.
Standardizing Substation Equipment. L. E. Cromer. (17) Mar. 31.
Standardizing Car Yard Layouts in Toronto.* (17) Mar. 24.
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Chicago Traffic, Value and Franchise. Methods used in Traction and Subway Valuation in Chicago. Travel for Unified System. Cost and Present Value of Elevated Lines. Summary of Financial Plan.* (17) Apr. 14.
Truck Cost Added to Detroit System Shows.* (17) Apr. 21.
Car Overhaul on Current.* H. L. Powers. (17) Apr. 21.

**Roads and Pavements.**

Old Macadam of Little Value as a Base for Concrete Pavement.* H. S. Thomas. (13) Mar. 1.
Tractive Resistance on Various Road Surfaces.* (86) Apr.
The Treatment of Wood Paving Blocks.* C. H. Teesdale. (60) Apr.
The Construction of Asphalt Pavements.* (60) Apr.
Standard Pavements for Los Angeles, Cal. (86) Apr. 4.
Broken Stone and Concrete Foundations for Pavements.* (86) Apr. 4.
Notes on the Maintenance of Gravel and Macadam Roads. (86) Apr. 4.
Method and Cost of Grading and Constructing Sidewalks on a Town Improvement.* R. E. Olson. (86) Apr. 4.
Methods and Cost of Screening and Washing Gravel for Country Road Construction. (86) Apr. 4.
Continuous Method of Maintaining Concrete Roads. (86) Apr. 4.
Methods and Cost of Cheaply Scarifying and Resurfacing Macadam.* Halbert P. Gillette. (86) Apr. 4.
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Dust Preventatives and Bituminous Binders.* G. Clarence Parker. (Paper read before Third Annual Conference on Road Construction.) (96) Apr. 5.
Points of Less in Hauling Road-Building Material; a Discussion of the Relative Merits of Teams, Tractors, Motor Trucks and Industrial Railways. A. H. Thompson. (13) Apr. 12.
Transportation of Materials.* Gustav Hoessler. (Paper read before Third Annual Conference on Road Construction.) (96) Apr. 12.
Procedure for Highway Lighting Districts. M. H. Wallhall. (14) Apr. 12.
The Contract and Specifications for Paving. Charles A. Mullen. (Paper read before the Dominion Good Roads Congress.) (96) Apr. 19.
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Bituminous Macadam Roads. Wm. G. Bulger. (Abstract of paper read before the Dominion Good Roads Congress.) (96) Apr. 19.
Tunnel Under Hudson Designed for Vehicular Traffic, Engineers Recommend that Tube between New York and New Jersey be Built by Floating Sections to Place and Sinking.* (14) Apr. 19.
Broken Old Concrete as Aggregate for New Paving Foundation. Clark R. Mandigo. (13) Apr. 19.
Monolithic Brick Pavement Tried in the Northwest. A. P. Cannon. (14) Apr. 19.
Unique Method of Handling Hot Oil on Maintenance Work. H. M. Belknap. (13) Apr. 26.
Weekly Reports Forms Simplify Highway Engineers' Work. M. W. Watson. (13) Apr. 26.
Good Construction Costs Lowered by Close Attention to Details.* Lester H. Morris. (13) Apr. 26.
How Much Water Should be Used in Road Concrete?* (13) Apr. 26.
Tests Show What Parts of City Streets are Used by Traffic.* H. T. Hutchins. (13) Apr. 26.
Correction of Alignments and Grades in Existing Highways. A. Fraser. (96) Apr. 26.
Road Oils and Tars. Arthur H. Blanchard. (Paper read before the Can. and Inter. Roads Cong.) (96) Apr. 26.
Road Foundation and Drains. R. Crawford-Muir. (Paper read before the Cong. on Road Construction.) (96) Apr. 26.
Les Travaux d'Ouverture de Routes et de Chemins en Algérie, Pendant la Guerre. Vialle. (43) Nov., 1916.

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**Sanitation.**

The Purification of Sewage in the Presence of Activated Sludge, Illinois. Edward Bartow, F. W. Mohlman and J. F. Schnellbach. (4) Feb.
Modern Sewage Treatment. T. Chalkley Hatton. (4) Feb.
Largest Drainage Pumping Plant is at Mattamuskeet.* (13) Mar. 1.
How the Pacific Electric Solves Surface Drainage Problems.* Clifford A. Elliott. (17) Mar. 17.
The Activated Sludge Process of Sewage Purification. Edward Ardern. (104) Mar. 23.
Opinions on Catchbasin Cleaning Methods Differ. (14) Mar. 24.
Tests Indicate Drain-Tile Beddings Increase Strength. (14) Mar. 24.
Activated Sludge Plant at Edmonton.* (From pamphlet by Geo. T. Hammond.) (96) Mar. 29.
Petrol Explosions in Sewers.* (12) Mar. 30.
Concrete Sewage Treatment Plant at Rochester, New York.* (67) Apr.
Relations between Sewage Disposal and Water Supply are Changing. George W. Fuller. (13) Apr. 5.
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# PAPERS AND DISCUSSIONS

JANUARY, 1917

Vol. XLIII. JANUARY, 1917. No. 1.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS

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Vol. XLIII. JANUARY, 1917. No. 1.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

By A. C. Dennis, M. Am. Soc. C. E.

To be Presented February 7th, 1917.

**Synopsis.**

**This paper describes the construction of the Rogers Pass Tunnel, on the Canadian Pacific Railway, through the Selkirk Range, in British Columbia.**

**The principal feature of the paper is the description of the pioneer, or auxiliary heading, method of construction, by which: adequate ventilation of the headings was secured; the removal of the muck was facilitated; the water, air, and ventilation pipes were not subject to disturbance; it was possible to expedite the work of enlargement of the main heading; a route for men, tools, and materials was provided; and the ventilation pipes were carried around the drilling, blasting, and mucking operations in the main tunnel, thus preventing delays due to gas and smoke. The "pioneer heading" appears to have justified its use for this tunnel.**

The Rogers Pass Tunnel is a double-track tunnel, slightly more than 5 miles long, being part of a local improvement, about 10 miles long, of the Canadian Pacific Railway Company's main line through the Selkirk Range, in British Columbia. The map and profile are shown on Fig. 1.

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Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The writer visited the site of this tunnel in the fall of 1912, at the request of the vice-president of the Canadian Pacific Railway, reported favorably on the economics of the proposed improvement, and suggested to the chief engineer that a line be tried locating the east portal in Bear Creek Valley, instead of Beaver Creek Valley, as then proposed. This line was located and adopted between the time of asking for bids and of letting the contract.

The late V. G. Bogue, M. Am. Soc. C. E., reported favorably on this tunnel improvement in connection with a very able and thorough report to the Canadian Pacific Railway on improvements to its lines from Calgary to Vancouver, surveys and studies for which were then being made by Mr. F. F. Busteed.

The writer, knowing that the time of construction of this tunnel would weigh heavily with the Railway Company in considering bids, gave considerable thought to devising some quick method of construction. The usual American method of driving a top heading and taking out the bench with a power shovel was obviously too slow, and the ventilation would be too difficult, for a tunnel of this length. The European method, of a bottom heading and stoping out the rest of the section, would be too expensive, under existing labor costs. Shafts or adits were impracticable. It was finally concluded that a working tunnel or "Pioneer Heading" entirely outside of the regular tunnel section would be economically justified, under the existing conditions, for the following reasons:

1.—The pioneer heading would serve as an intake for forced circulation of fresh air through the cross-cuts and out of the main tunnel, enabling the work to be resumed immediately after blasting the main tunnel enlargement.

2.—It would serve to take muck from the headings around the drillers, blasters, and from shovel operations in the enlargement of the main tunnel.

3.—It would serve to conduct water, air, and ventilation pipe lines to the headings, so as to be undisturbed by the enlargement workings.

4.—It would make it possible to drill the main heading for enlargement far ahead of, and without interference from or with, the main tunnel blasting or mucking for enlargement.

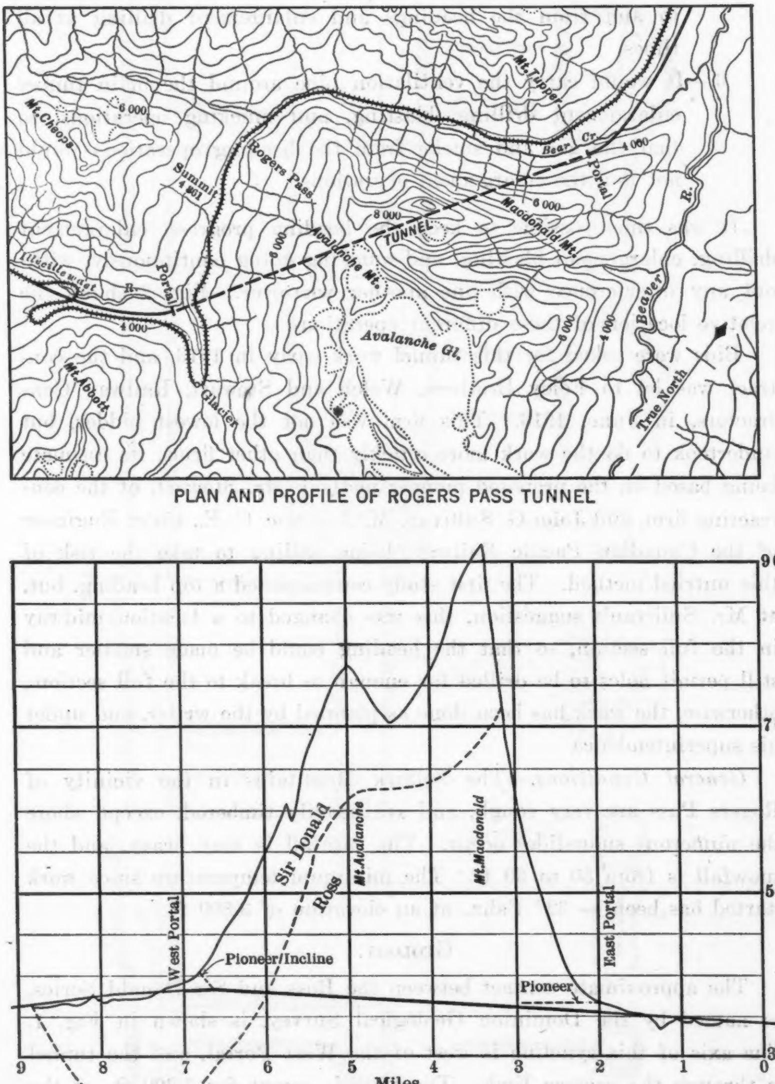

Mt. Cheops
6 000
Rogers Pass
Summit
Mt. Tupper
Bear Cr.
4 000
Portal
Macdonald Mt.
6 000
8 000
TUNNEL
Avalanche Mt.
8 000
Illecillewaet R.
Portal
4 000
Avalanche Gl.
Mt. Abbott
Glacier
Beaver R.
4 000
6 000
True North
PLAN AND PROFILE OF ROGERS PASS TUNNEL
9 000
7 000
5 000
3 000
Sir Donald
Ross
Mt. Avalanche
Mt. Macdonald
West Portal
East Portal
Pioneer/Incline
Pioneer
9
8
7
6
5
4
3
2
1
0
Miles

FIG. 1.

5.—It would supply a route for men, tools, and materials to pass to and from the headings and enlargement drilling at all times.

6.—It would carry the ventilation pipe around the main tunnel enlargement drilling, blasting, and mucking operations, so that the gas and smoke from the heading operations would not foul the enlargement operations.

It was thus possible to keep the heading progress, enlargement drilling, enlargement blasting, and mucking going continuously, without any interference with one another whatever. Fig. 2 shows the relative location of these different operations.

Bids were asked for this tunnel work early in 1913, and the contract was let to Foley Brothers, Welch and Stewart, Railway Contractors, in June, 1913. This firm was not the lowest bidder, but undertook to do the work more quickly than other firms, its estimate being based on the proposed pioneer method, Mr. Stewart, of the contracting firm, and John G. Sullivan, M. Am. Soc. C. E., Chief Engineer of the Canadian Pacific Railway, being willing to take the risk of this untried method. The first study contemplated a top heading, but, at Mr. Sullivan's suggestion, this was changed to a location midway in the full section, so that the heading could be made smaller and still permit holes to be drilled far enough to break to the full section; otherwise, the work has been done as planned by the writer, and under his superintendence.

*General Conditions.*—The Selkirk Mountains in the vicinity of Rogers Pass are very rough, and are heavily timbered, except where the numerous snowslides occur. The rainfall is very heavy, and the snowfall is from 30 to 50 ft. The minimum temperature since work started has been — 32° Fahr., at an elevation of 3 800 ft.

## GEOLOGY.

The approximate contact between the Ross and Sir Donald Series, as named by the Dominion Geological Survey, is shown in Fig. 1. The axis of this syncline is west of the West Portal, and the tunnel is through the eastern limb. The tunnel, except for 1 200 ft. of the east end and 400 ft. of the west end, is all in solid rock, classified as quartzite in the geological reports, but consisting largely of schists. The rocks contain no fossils, and cannot be assigned definitely to any

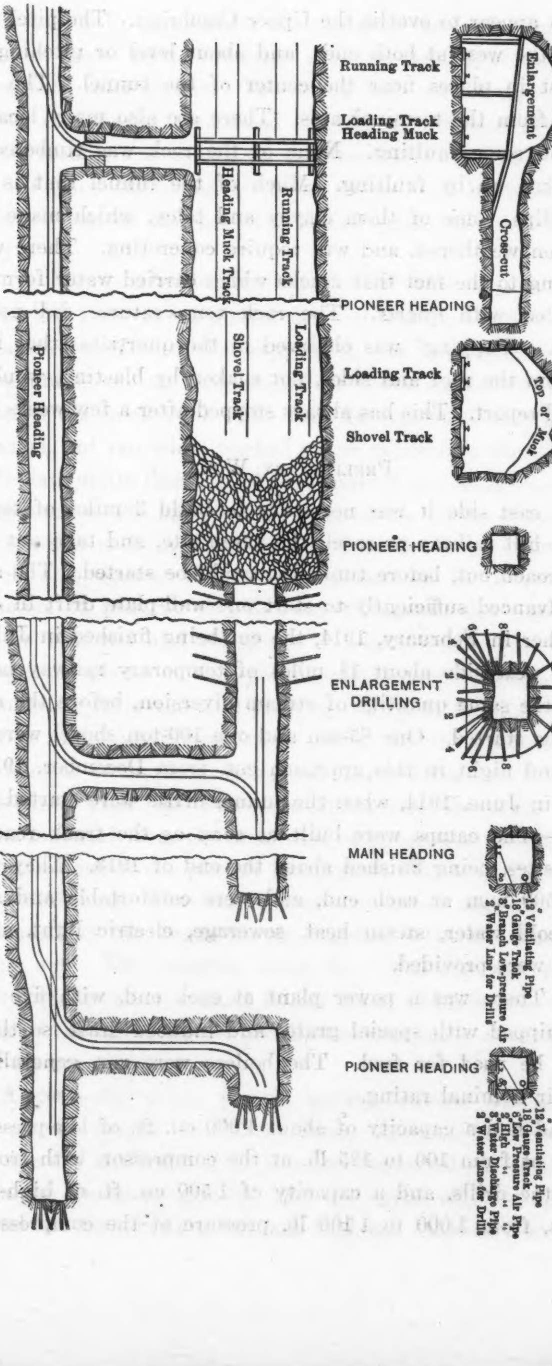

Running Track
Loading Track
Heading Muck
Enlargement
Cross-cut
Running Track
Heading Muck Track
PIONEER HEADING
Loading Track
Shovel Track
Top of Muck
Loading Track
Shovel Track
Pioneer Heading
PIONEER HEADING
ENLARGEMENT
DRILLING
MAIN HEADING
12" Ventilating Pipe
18" Gauge Track
8" Branch Low-pressure Air
2" Water Line for Drills
PIONEER HEADING
12" Ventilating Pipe
18" Gauge Track
8" Low Pressure Air Pipe
2" High " " "
3" Water Discharge Pipe
2" Water Line for Drills

FIG. 2.

period, but appear to overlie the Upper Cambrian. The pitch is rather steeply to the west at both ends, and about level or pitching slightly to the east in places near the center of the tunnel. The strike is about 60° from the terminal axis. There are also many local irregularities and some faulting. None of the rock was timbered, except where broken up by faulting. Much of the tunnel that is in hard rock has slips, some of them clayey and talcy, which make the roof unsafe when weathered, and will require concreting. There was little water, owing to the fact that cracks which carried water formerly had become filled with quartz. The rock temperatures did not exceed 75 degrees. "Popping" was observed in the quartzite; that is, pieces of rock from the roof and sides, not shaken by blasting, would fly off with a loud report. This has always stopped after a few weeks exposure to the air.

## Preliminary Work.

On the east side it was necessary to build 3 miles of temporary, heavy, side-hill railway to reach the camp site, and take out a rather heavy approach cut, before tunneling could be started. The approach cut was advanced sufficiently to start one wall-plate drift in January, and the other in February, 1914, the cut being finished in July.

On the west side about 1½ miles of temporary railway had to be built, and the same quantity of stream diversion, before the approach cut could be started. One 85-ton and one 100-ton shovel were worked both day and night in this approach cut, from December, 1913, until the finish, in June, 1914, when the tunnel drifts were started.

*Camps.*—The camps were built as soon as the track reached the respective sites, being finished about the end of 1913. They provided room for 500 men at each end, and were comfortable and sanitary. Hot and cold water, steam heat, sewerage, electric light, and good ventilation were provided.

*Plant.*—There was a power plant at each end, with five 150-h. p. boilers, equipped with special grates and induced draft, so that slack coal could be used for fuel. The boilers were run generally much beyond their nominal rating.

Each end had a capacity of about 4 000 cu. ft. of low-pressure air, that is, air at from 100 to 125 lb. at the compressor, with from 90 to 100 lb. at the drills, and a capacity of 1 500 cu. ft. of high-pressure air, that is, from 1 000 to 1 100 lb. pressure at the compressors, and

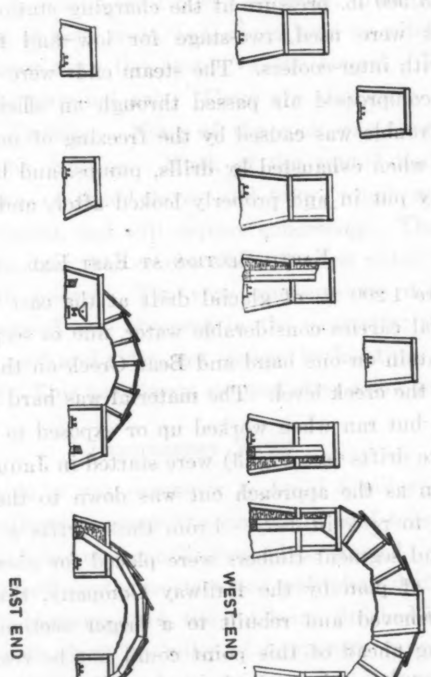
from 800 to 900 lb. pressure at the charging stations. Ingersoll-Rand compressors were used, two-stage for low and four-stage for high pressure, with inter-coolers. The steam ends were compound condensing. The compressed air passed through an efficient after-cooler, so that little trouble was caused by the freezing of moisture in the compressed air when exhausted by drills, pumps, and blowers. The plant was properly put in and properly looked after, and caused practically no delay.

### Earth Section at East End.

There are 1 200 ft. of glacial drift at the east end of the tunnel. This material carries considerable water, due to seepage from the side of the mountain on one hand and Bear Creek on the other, the tunnel being below the creek level. The material was hard and compact when first opened, but ran when worked up or exposed to the air.

Wall-plate drifts (see Fig. 3) were started in January and February, 1914, as soon as the approach cut was down to the wall-plate grade, and finished to rock in July. From these drifts a stope was started in March, and segment timbers were placed for about 100 ft. Owing to a change of plan by the Railway Company, this segment timber had to be removed and rebuilt to a larger section, so that stoping and timbering ahead of this point could not be resumed until June. The timbering from the wall-plate up was completed in October. A locomotive crane was found very useful for taking the timber and other material from the framing yard and delivering it in the wall-plate drifts. After the wall-plate drifts met the main heading, in July, the material from the break-ups or stopes was hauled to the first cross-cut and out of the pioneer heading, so that it would not interfere with the timbering. Previous to that time it was dumped at the portal and reloaded by the steam shovel, together with the material of the approach cut. The material from the break-ups was passed down through a chute into the small tunnel cars. Considerable turning and crushing of the wall-plate occurred, necessitating a great deal of propping of the segments.

In August, the steam shovel, having finished the approach cut, started on the tunnel excavation, being operated by compressed air. During the day shift, the wall-plate was supported on temporary batter posts as the shovel progressed, and the night shift straightened the sides and set the plumb posts and lagging. During the night shift,




METHOD OF SOFT GROUND TUNNELING

FIG. 8.

the locomotive crane delivered the material required for the segment timbering the next day. After being impressed by the crushing of the wall-plate and segment timbers, it was with considerable anxiety that the undermining of the wall-plate with the power shovel was attempted. The characteristic of the ground, of remaining firm for a short time after being opened up, was encouraging. As it turned out, the fear of loosening part of the tunnel by the side sliding in, while the wall-plate was carried on batter posts, was groundless. There was no settlement of the wall-plate of more than 1 in., and the work was hastened and cheapened by this method much more than by driving drifts to place posts before excavating the core of the tunnel. The excavation of this earth section was finished in December, 1914.

## Earth Section at West End.

There were about 400 ft. of soft ground at the west end. The west portal was under a swamp, difficult to drain, and at the intersection of two creeks which have both been diverted; but this soft ground section was at all times saturated with water, in spite of efforts at surface draining. The material was very fine sand and clay, with large boulders, and closely resembled quicksand or wet concrete. One boulder was large enough to drive the crown drift through. The cracks between the breast boards or poling had to be stopped with hay in order to prevent the material from running into the heading. This material was very heavy, and the small drifts had to have sets about 2 ft. apart, and additional posts under many of the caps. Boulders near the sides were jacked back into the quicksand in order to clear the timbers. The method of opening up this end was entirely different from that at the east end, as shown by Fig. 3. The bottom heading of the drifts for the plumb posts was started first, with a view to drainage, and was followed closely by the top heading. The muck from the top heading was trapped into cars in the bottom heading.

The crown drift was driven about even with the top heading of the plumb-post drifts, and seemed to be the most difficult of all to hold, due to the disturbance of the ground by the other drifts. When the plumb-post drifts had been driven to where the rock reached the level of the top of the plumb posts, the latter were discontinued and replaced for the remainder of the distance by wall-plate drifts, from

which a cross-cut was made to the main heading when the full rock section was reached.

After the plumb-post drifts were finished, the plumb posts, being double 12 by 16-in. timbers, were all set and thoroughly blocked against the timbering in the drifts, and the wall-plate was set on the rock beyond the posts. A break-out was started in the sides of the crown drift, about 100 ft. from the portal, and the timber segments were put in, working both ways from this break-out. The muck from break-outs was used to back-fill the post drifts. The posts and segments were set leaning away from the portal, and iron bars were spiked on the segments in order to resist the thrust of the material toward the portal. Other break-outs were started farther in, and segments were set under the crown bars, trued up, and packed thoroughly by timber between the segments and the timber supports of the outside segments. No wall-plates were used, and the segments were generally 32 in. thick. Work in this soft ground section was very slow and expensive. The time required for the 400 ft. was from June, 1914, to February, 1915. The excavation with the power shovel followed the timbering operations. There was some uncertainty as to whether or not bracing would be necessary to prevent the side pressure on the plumb posts from shoving them in after the excavation had removed the earth resistance on the tunnel side of these posts; but no movement occurred, and no bracing was needed. The material sets or hardens when drained.

## East Pioneer Heading.

The east pioneer heading was started in September, 1913, about 50 ft. north of the main tunnel, 700 ft. west of the east portal, and about 60 ft. above the main tunnel level. This location was adopted in order to save 700 ft. of pioneer tunneling, to reduce the quantity of soft ground heading, to enable work on the heading to start sooner than that on the approach cut, and to get rid of the muck readily. The power was furnished by the temporary erection of an old compressor along the Canadian Pacific Railway track above and a pipe line down the hill to the work. This heading was run as nearly level as drainage would permit. The grade of the main heading reached the grade of the pioneer at the third cross-cut, the two former cross-cuts being driven to the dip, and material from the main heading being hoisted up the incline. The heading reached solid rock about 600 ft. in,

at which point the first inclined cross-cut was started, at about the beginning of 1914. The pioneer tunnel, in rock, was driven about 2 miles in 1½ years. The maximum monthly progress was 776 ft. The daily average was 20 ft. for the entire drift in rock.

### West Pioneer Tunnel.

The west pioneer heading was started by an incline, 300 ft. long, from the rock outcrop, 700 ft. east of the west portal, about 150 ft. above the main heading level, and 50 ft. south of the main tunnel line. This location was selected in order to provide dumping ground, shorten the length of heading to be driven, avoid soft ground tunneling, and permit an earlier beginning than by waiting for the approach cut excavation. This incline was very wet and took 2 months to drive, being finished in the latter part of July, 1914. This pioneer tunnel was driven for a length of more than 1½ miles in less than a year, the maximum monthly progress being 932 ft., which is far beyond any previous American record known to the writer. The daily average of 24 ft. for nearly a year, largely through very hard quartzite is also unusual.

### Pioneer Headings in General.

The pioneer tunnel, in rock, was 7 ft. high and 8 ft. wide. It was driven with light hammer drills, using hollow steel, with water attachments. Three drills, in general, but four in the hardest rock, were used in a heading. Spare drill machines, for the replacement of drills out of order, were kept conveniently at hand in the heading. No repairs were made under ground. The hammer drills are convenient and rapid, the delay and expense of their constant breakage perhaps balancing the advantage of speed under ordinary conditions. The drills are mounted on a light horizontal bar, about 18 in. below the roof line. Air and water are taken over the muck pile, or on hooks in the side, by a single hose line for each, to a manifold from which short individual hose lines supply the drills.

Light cars (½ cu. yd.) were used for muck, and the latter was taken off the track, instead of building sidings for this purpose. Shoveling plates were used at the face and on the side away from the track for some distance back of the face, in order to facilitate the handling of empty muck cars. The ventilating pipe was a 12-in. wooden water pipe connected to the Connersville blowers used for the exhaust.

This pipe was hung on the side away from the track, close up to the roof and was carried to within 20 ft. of the face. Little damage was done to this pipe by blasting. The blowers were started exhausting when the first shot was fired, or a little before, and were run for 20 min. The men got back to work in from 5 to 10 min. No compressed air was allowed to be blown out for ventilating purposes. After a round was shot, the drillers followed the smoke back, barring down the roof, bringing explosives to re-shoot, and wetting down the muck pile, sides, roof, and face with water hose. The muckers cleared the track and began loading the muck which was scattered back.

When no further blasting was required, the lights were hung, the foreman sighted the line and grade point in the face, and the drilling gang set up the horizontal bar, placed their drills and proceeded. There was rarely any muck to be handled before the drilling could be started, as it was thrown back from the face by the heavy loading in the bottom holes and the fact that they were shot last, for this purpose. There were two helpers to three drills, and they brought up and changed the steel and adjusted the drill machines. When the drilling from the upper set-up was completed, the drillers took down the machines and carried them back, with the hose connections still attached, and oiled them up. After the mucking was done, the bar was dropped to the lower set-up, near the floor, and the drills were set to drill the bottom holes or lifters. The drills were carried forward, put on the bar, and were drilling sometimes in less than 2 min. after the bar was dropped. While the bottom holes were being drilled, the muckers laid the track, adjusted and covered the mucking sheets with muck, and brought up the explosives. The holes were loaded by the machine men, helpers, and foremen.

For the small part of the tunnel where re-shooting was not necessary, an 8-hour shift could do two rounds per shift, or a little better. Two men pick down the muck, and three men load the car and push it out, while three others stand by with an empty car, ready to put it on the track and load it. The three men taking out the loaded car return near the face with an empty car, take it off the track, and rest until the load comes out. The men get a rest from the monotony of steady continuous shoveling, and the empty car is available at once after the load goes back. The pipes for ventilating, and

FIG. 4.—BEGINNING DRILLING FROM UPPER SET-UP OF BAR IN THE PIONEER HEADING.

FIG. 5.—MAIN HEADING, SHOWING ENLARGEMENT DRILLING.

for air and water were laid by a pipe man and helper, who looked after several headings.

Doing this work with muckers was unsatisfactory. Muck cars were taken from the heading back to a siding by a single mule, and from there to the dump by a two- or three-mule team driven tandem, until this method became inadequate, and then compressed-air locomotive haulage was substituted for the long haul. The heading muck cars, after the shovel and switching track had cleared a cross-cut, were taken to the cross-cut, pulled up an inclined trestle by air hoist and cable, and dumped into standard-gauge cars, as shown by Fig. 2. The cross-cuts are from 1 500 to 2 000 ft. apart, as shown by Fig. 6. Air pressure was maintained at about 90 lb. at the drills, which required 125 lb. at the compressors toward the end of the work.

The rounds were usually 6 ft. The cut holes were generally shot once or twice, and the remainder of the cut was shot with the rest of the round. All shooting in headings was done with fuse. The explosives used were 40 and 60%, low-freezing gelatine, with No. 8 caps. The rock was hard to break, and the quantity of explosives was necessarily high. From 21 to 28 holes were drilled in the pioneer face. Change of shifts was made at the heading, the shift coming on taking the tools out of the hands of the shift finishing. Three shifts a day were worked every day in the year, except for one day at the east end, due to the burning of the fan house, and one day due to the breaking of the air main by a snowslide. The pioneer gang drove the cross-cuts between the pioneer and the main tunnel heading. The pioneer tunnel was not driven for the last mile, connection being made by the main heading only, which was all drilled up for enlargement before the enlargement blasting reached this section. The main heading work had to be completed before the enlargement blasting and mucking reached the last cross-cut, as it would have been impossible to maintain the air connections, or ventilate the main heading, after that time, so as to allow continuous work.

### Main Heading.

The main heading, generally known on the work as the "Center Heading", was entirely through the rock section. It was 11 ft. wide and 9 ft. high, the center line being the same as that of the com-

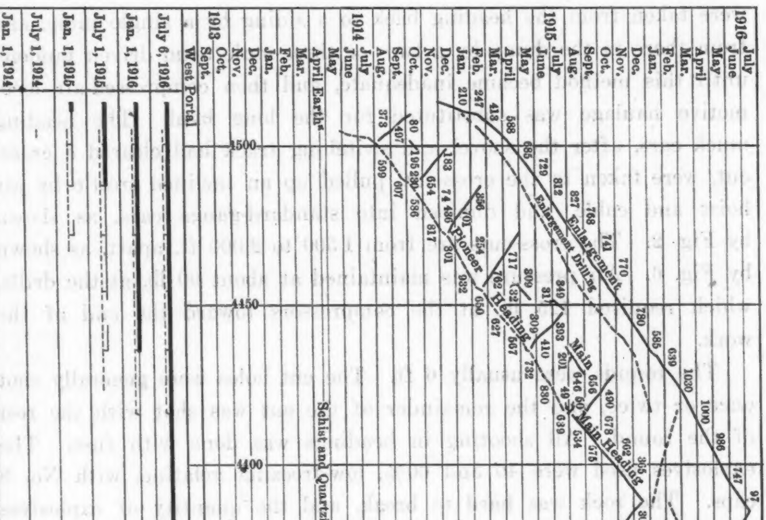

West Portal
East Portal
Enlargement
Enlargement Drilling
Pioneer Heading
Main Heading
Schist and Quartzite Rock
April Earth
Earth
4500
4450
4400
4350
4300
4250

FIG. 6.

pleted tunnel and the bottom being 6 ft. above the sub-grade. The position and size were such that lateral holes could be drilled from this heading to break the enlargement to the required dimensions. The air, water, and ventilating pipes for this heading were branches from the mains laid in the pioneer heading. Access to this heading was obtained through the cross-cuts from the pioneer, and muck was handled around the enlargement operations by the pioneer route. This heading was generally driven in a westward direction, on account of the drainage. The system of driving was similar to that in the pioneer. The rounds averaged about 7 ft., and 32 holes were drilled in the hardest rock. The main heading was sometimes driven from several faces. The progress is shown by Fig. 6. The average daily progress per heading at the east end was slightly more than 16 ft., and the maximum monthly progress was 621 ft. The average daily progress per heading at the west end was 20 ft.; the maximum monthly progress was 762 ft.

### Headings in General.

The headings were sublet at a price per foot and a bonus for more than 450 ft. per month, the sub-contractor furnishing the labor and explosives only. This arrangement proved unsatisfactory, and was discontinued in September, 1914. After this time a substantial bonus, based on the monthly footage and equated for hard rock, was given, and divided among all men directly connected with the heading driving, in proportion to their regular wages earned for the month. It was agreed that the rate of bonus would not be reduced. The latter arrangement resulted in 23% greater speed, and a large saving in compressed air and other items furnished to the sub-contractor under the former arrangement.

### Enlargement.

*Drilling.*—The enlargement drilling, after some experimenting, was done as shown by Figs. 2 and 7. Each hole was pointed by clinometer, the column carrying the drill being set always at the same distance off the center line, and the arm for the lower and upper sets being always the same distance above the sub-grade. Line and levels were furnished by the Railway Company's engineers, and a string was stretched by which the columns and arms were located. Each drill hole had its proper distance from the arm. The drill

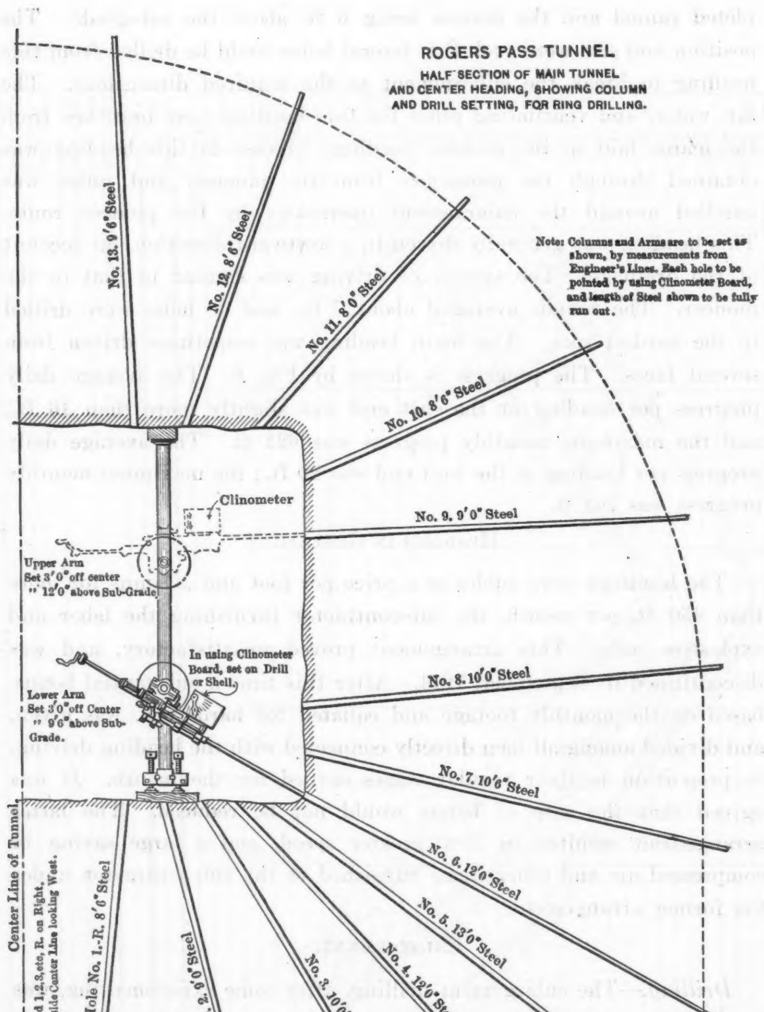

ROGERS PASS TUNNEL
HALF SECTION OF MAIN TUNNEL
AND CENTER HEADING, SHOWING COLUMN
AND DRILL SETTING, FOR RING DRILLING.
Note: Columns and Arms are to be set as shown, by measurements from Engineer's Lines. Each hole to be pointed by using Clinometer Board, and length of Steel shown to be fully run out.
No. 13. 9'6" Steel
No. 12. 8'6" Steel
No. 11. 8'0" Steel
No. 10. 8'6" Steel
Clinometer
No. 9. 9'0" Steel
Upper Arm
Set 3'0" off center
" 12'0" above Sub-Grade
In using Clinometer Board, set on Drill or Shell.
No. 8. 10'0" Steel
Lower Arm
Set 3'0" off Center
" 9'6" above Sub-Grade.
No. 7. 10'6" Steel
No. 6. 12'0" Steel
No. 5. 13'0" Steel
No. 4. 12'0" Steel
No. 3. 10'0" Steel
No. 2. 9'0" Steel
Hole No. 1.-R. 8'6" Steel
Center Line of Tunnel
Holes numbered 1, 2, 3, etc., R. on Right, and L. on left side Center Line looking West.
225 Lin. ft. per round.

FIG. 7.

holes were thus bottomed at a regular distance beyond the neat line of the completed excavation. The holes, being bottomed with reference to the line and grades given by the engineers, were not affected by irregularities in the heading driving. The columns were set by men for that purpose, so that the drillers and helpers had only to do the drilling. The drill steel was brought to the drillers, and the dull steel was taken away. The drillers and helpers were paid their wages in any event, but the footage for each man was kept, and if the price set per foot drilled amounted to more than his wages, he was given the difference as a bonus check. Air and water connections were made for every third ring of holes, and only one drill machine, though handled by each runner of the three daily shifts, completed the three rings, and then moved to the head of the line, taking the next three rings. Congestion of men and material was thus avoided, and each man had a fair chance to work on an equal quantity of hard and soft rock. There was extreme variation in the quantity drilled by different men and in different rock. The same man might do only 6 ft. a shift in the hardest quartzite, and more than 100 ft. per shift in the softer schist. New men, after a month's practice, generally made more footage than men of long experience in mining. In general, it was found better to train green men than to try to get men accustomed to piston drills to learn to run hammer drills. Most of the rings were 6 or 6½ ft. apart. When explosives rose in price it was found economical to space the rings 5 ft. apart, as the extra drilling cost was balanced by the saving in explosives, with the added advantage that the muck was broken into smaller pieces and scattered farther back. Where the roof was soft and full of slips, so that trouble was anticipated, the upper set of arms on the column was lowered 1 ft., in order to leave some trimming of the roof to be done by jack-hammer, flat holes and light blasting. The air and water for the enlargement drilling, as well as the supplies, came by the pioneer tunnel and the cross-cuts, so that this drilling was not disturbed by the enlargement blasting. The drilling for the last mile, where no pioneer tunnel was driven, was started at the middle and progressed toward the portal, the track, pipe, etc., being removed as the drilling was finished. The stopping of the pioneer tunnel was well-timed, as the main heading was driven and the enlargement

drilling completed just in time to avoid delaying the enlargement blasting and mucking at the east end.

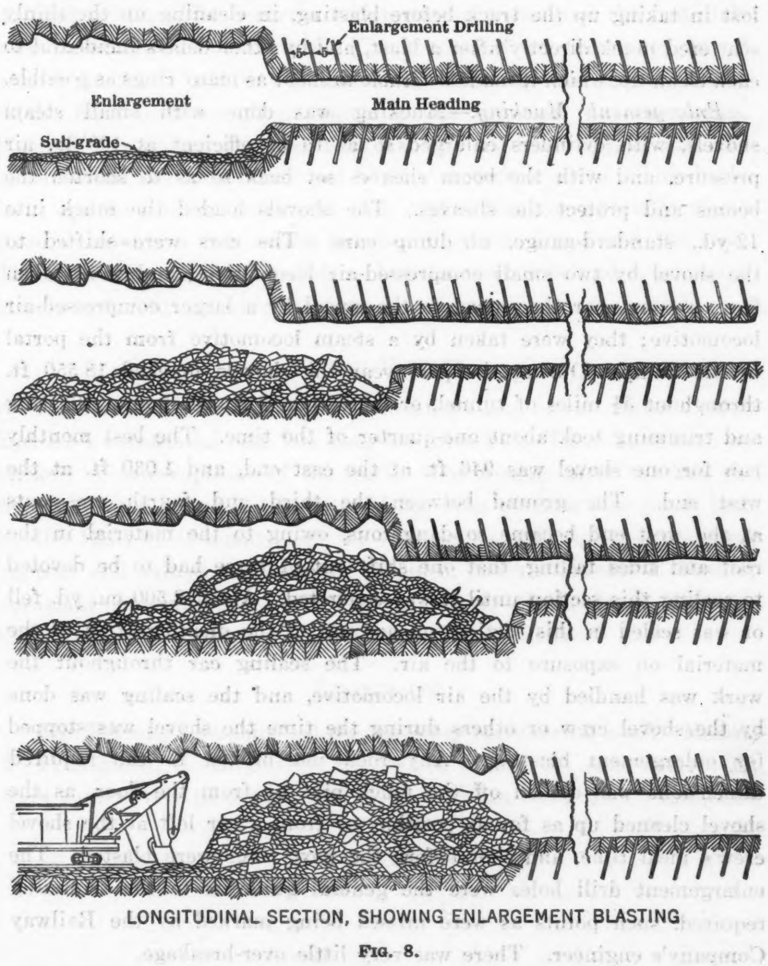
*Enlargement Blasting.*—There was considerable difficulty in breaking the bottom to sub-grade when the rock excavation was first started. This was overcome by dropping the floor of the main heading 1 ft. and drilling the holes in the bottom 1 ft. deeper. Difficulty was found also in getting the sides below the springing line to break for the full width. This was overcome by drilling one or two relief holes at this locality in tough breaking rock. In tough rock, two, four, six, and sometimes eight, holes were sprung. (Fig. 2.) If oversprung, the ring being shot was likely to break into the next ring and explode it, or shake it up so as to spoil the effect of the blast.

Generally, from ten to fifteen rings were kept loaded ahead. Any part of a hole which had not broken, and could be found, was reloaded and shot with the next ring. Generally, a little muck was left in the face by the power shovel in order to prevent the first ring from scattering back too far. If the previously shot material had not broken to the required width, however, all the muck was loaded, and jack-hammers were used to drill up this tight rock, after which it was shot before the regular rings were blasted. Several bottom rings were first blasted, then a top and bottom ring were blasted together until the muck piled up to within 4 or 5 ft. of the roof. Then blasting was discontinued, and the men scaled and trimmed the roof, working from the muck pile. (Fig. 8.) Where no holes had to be reloaded, rings could be blasted at intervals of from 15 to 20 min. The blasting was done with a battery in the main heading, and the bottom holes were all loaded ahead, the wires being wound up and stuck in the holes, from which they could readily be pulled out and connected. The upper holes were loaded, but no primers were put in until ready to blast. The holes were loaded to within 4 ft. of the collar, whether sprung or otherwise.

When retiring in the main heading to blast, the blasting gang took back the scaling tools, so that they might examine and scale the roof of the heading if necessary as they returned. After several rings had been blasted, the power shovel crew commenced to clean up the beginning of the muck heap, and only retired a few minutes for the following blasts. Several top rings were generally held and shot at meal times, when the shovel had excavated sufficient muck to

provide room for more without blocking the airway and manway over the pile.

The smoke and gas from the blasting were quickly taken out by the fresh air forced into the pioneer tunnel by a "Sirocco fan" at the




LONGITUDINAL SECTION, SHOWING ENLARGEMENT BLASTING

FIG. 8.

portal. The air circulated through the pioneer tunnel and the cross-cut ahead of the blasting, and then back through the main heading, over the muck pile, and out at the portal of the main tunnel. This circulation was prevented from short circuiting by stoppings and doors in the

cross-cuts passed by the shovel. The quantity possible to shoot depended on the distance the muck was thrown back, or the quantity of muck for which there was room without interfering with the scaling and trimming of the roof; it varied from 24 to 110 ft. Much time was lost in taking up the track before blasting, in cleaning up the thinly scattered muck directly after a blast, and by other delays incidental to each clean up, which it made desirable to shoot as many rings as possible.

*Enlargement Mucking.*—Mucking was done with small steam shovels, with cylinders enlarged so as to be efficient at 100 lb. air pressure, and with the boom sheaves set back so as to shorten the booms and protect the sheaves. The shovels loaded the muck into 12-yd., standard-gauge, air-dump cars. The cars were shifted to the shovel by two small compressed-air locomotives, and were taken from a spur near the shovel to the portal by a larger compressed-air locomotive; they were taken by a steam locomotive from the portal to the dump. During the past year the shovels mucked 18 550 ft. throughout 3½ miles of tunnel, or more than 2 ft. per hour. Blasting and trimming took about one-quarter of the time. The best monthly run for one shovel was 946 ft. at the east end, and 1 030 ft. at the west end. The ground between the third and fourth cross-cuts at the west end became so dangerous, owing to the material in the roof and sides falling, that one shift out of three had to be devoted to scaling this section until it was concreted. About 2 500 cu. yd. fell or was scaled in this section, on account of the disintegration of the material on exposure to the air. The scaling car throughout the work was handled by the air locomotive, and the scaling was done by the shovel crew or others during the time the shovel was stopped for enlargement blasting. Any rock not broken to the required dimensions was drilled off the muck pile, or from the floor, as the shovel cleaned up as far as possible, or from a car left at the shovel crew's meal time, and shot before the next rings were blasted. The enlargement drill holes were the general guide as to the trimming required, such points as were missed being marked by the Railway Company's engineer. There was very little over-breakage.

## Concreting.

About 1½ miles of the tunnel, including the soft ground at each end, required concreting. This work was sublet to the Bates and

Rogers Construction Company, of Chicago and Spokane. The concrete section is heavily reinforced, which prevented the use of the pneumatic method of placing. The sub-contractors used wooden forms, and deposited the concrete from a platform near the roof reached by an inclined trestle. The concrete mixer was on a car, and the materials were on other cars back of it. The concrete from the mixer flowed into a small car which was hauled by cable up the trestle incline to the high platform, from which it was shoveled into the forms. Much of the lining required back as well as front forms, and the space behind the back forms was filled with rock or wood. This back-form and back-filling work was slow and expensive, especially where there were only a few inches between the back forms and the rock. The concreting has been well done, but the work is behind time, owing largely to conditions which the sub-contractors could not control.

## General Results.

The tunnel has been finished 11 months ahead of the contract time, and for a substantial sum less than the price bid. There have been no strikes, or interruptions of the work. The management has had the necessary money and authority, and has been given a free hand, both by the Railway Company and the contractors, and has had the loyal support and valued assistance of the men, notably Messrs. J. Fowler, Assistant Superintendent, T. Truran, Master Mechanic, J. Roberts, Chief of Office Force, and M. C. Brian, Shovel Runner.

## Engineering.

The Railway Company did the necessary engineering work. Messrs. F. F. Busteed, H. G. Barber, and W. A. James, M. Am. Soc. C. E., were successively in charge, Mr. J. G. Sullivan being Chief Engineer.

to be lightened upon trucks, or [illegible] and [illegible]. The continuous section is [illegible] which permitted the use of the [illegible] method of placing. The [illegible] and deposited the concrete from a platform near the [illegible] reached by an inclined road. The concrete mixer [illegible] and the [illegible] was [illegible]. The concrete [illegible] the timber forms [illegible] which was hoisted by cable to the [illegible] to the high platforms, from which it was [illegible] into the forms. Much of the [illegible] as well as [illegible] forming, and the [illegible] and the back forms were [illegible]. The [illegible] form and [illegible] work [illegible] and [illegible] especially [illegible] there were [illegible] a few [illegible] the back forms and the [illegible]. The [illegible] has been well done, but the work [illegible] and times [illegible] largely to conditions which the [illegible] could not control.

General Remarks.

The tunnel has been finished 12 months ahead of the contract time, and [illegible] a substantial [illegible] less than the price bid. There have been no strikes, or [illegible] of the work. The management has had the necessary money and [illegible], and has been given a free hand, [illegible] by the Railway Company and the [illegible], and has had the loyal support and valued assistance of the men, notably Messrs. J. [illegible], Assistant Superintendent; T. Byrne, Master Mechanic; J. [illegible], Chief of [illegible] and J. G. [illegible], Shovel Runner.

Engineering.

The Railway Company did the [illegible] work. Messrs. [illegible] H. [illegible] and W. A. [illegible], [illegible] were [illegible] Mr. J. G. Sullivan being Chief Engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## THE WATER SUPPLY OF PARKERSBURG, W. VA.

BY WILLIAM M. HALL, M. AM. SOC. C. E.

TO BE PRESENTED FEBRUARY 21ST, 1917.

### SYNOPSIS.

**This paper describes the process of selecting a new water supply for the City of Parkersburg, W. Va., the investigation of the possible sources of supply, especially the natural underground supply in the neighboring Ohio River bottoms and plateau, the physical and geological phenomena relating thereto, the methods considered for collecting the water for pumping, and the novel method finally adopted, which consists of an infiltration system composed of strainer pipes laid in the bed of the river and overlaid with a bed of washed gravel and sand.**

### INTRODUCTION.

The City of Parkersburg, W. Va., (population 17 842 in 1910) constructed works for a public water supply in 1884. The pumping station was established on the banks of the Ohio River, and for more than 25 years the supply was drawn from the muddy and polluted waters of that stream, and used for all purposes without any treatment whatever. With increasing quantities of sewage entering the river from Pittsburgh, Wheeling, and other communities in the Ohio Valley above Parkersburg, this supply became increasingly unsatisfactory. Typhoid fever was quite prevalent, and investigations in 1906 and

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

1907 indicated for that disease annual death rates of 75 and 60, respectively, per 100 000 population.

In 1907 a law was enacted creating for Parkersburg a Water-Works Commission, composed of three city officials, including the Mayor, and four citizens, of whom the writer was one.* This Commission instituted investigations for the purpose of determining the best method of treatment of the Ohio River water, if that should continue to be the supply, and whether or not water could be obtained from wells, cribs, or from a gravity source. As a part of the project, consideration was also given to a new pumping station to supersede the original plant, which had suffered much from wear and tear in pumping muddy water for such a long time. It is not within the scope of this paper to describe the details of the pumping station, distributing reservoirs, pipe lines, etc.

### Preliminary Investigations.

Morris Knowles, M. Am. Soc. C. E., was engaged to make recommendations relative to a new water supply. In addition to gravity and inpounding sources of supply and filtration of the Ohio River water, he investigated the feasibility of obtaining a well-water supply from the Camden Farm, which lies immediately north of the upper city limits of Parkersburg. This is shown by Fig. 1.

Mr. Knowles reported in August, 1908, for a recommended 4 000 000-gal. daily supply, that, although the well proposition was attractive, further investigation should be made, perhaps in other localities, or somewhat removed from the Camden Farm. His test of a 12-in. well did not indicate a serious lowering of the ground-water level, but, for safety, a more elaborate test was recommended, as it was realized that a large factor of safety should be required in establishing a well plant. Based on his recommendations, the citizens voted on April 22d, 1909, to authorize a bond issue of $270 000 to build the proposed new water-works; and he was authorized to prepare plans and specifications for the construction thereof.

Some doubt as to the permanency of the well-water supply arose in the minds of some of the city officials, and, on request, further

* The Commission consisted of Messrs. W. D. Pedigo, Mayor, C. D. Forrer, of the Board of Affairs, Walter Gerwig, Councilman, and the following citizen members: Messrs. S. D. Camden, H. H. Moss, B. S. Pope, and the writer. Mr. Camden was elected President of the Commission, and Mr. Forrer, Secretary. After the expiration of Mr. Forrer's term of office, Mr. George P. Chase, Attorney, though not a member of the Commission, served as Secretary.

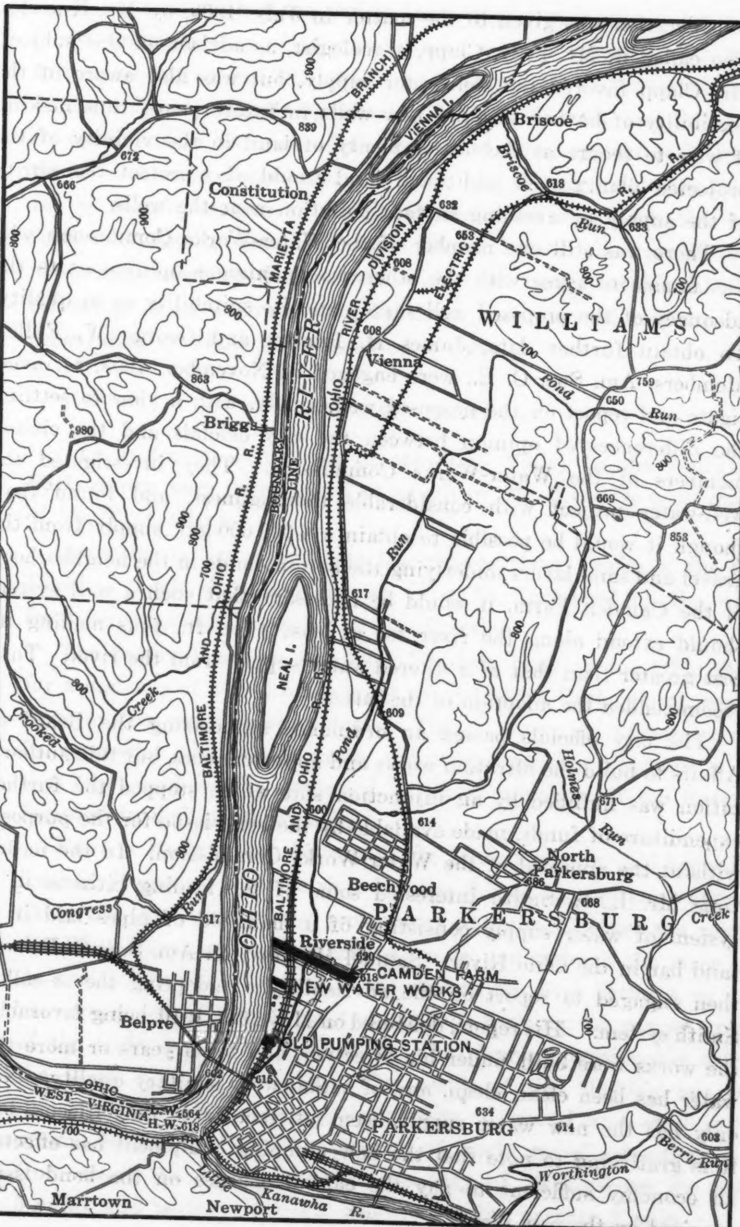

Briscoe
Briscoe
Run
R.
Constitution
MARIETTA
BRANCH
VIENNA I.
RIVER
DIVISION
ELECTRIC
WILLIAMS
Vienna
Pond
Run
Briggs
R. R.
BOUNDARY LINE
OHIO
RIVER
NEAL I.
BALTIMORE AND OHIO
Pond
Run
Crooked Creek
Holmes
Run
North Parkersburg
Beechwood
PARKERSBURG
Creek
Congress
Run
OHIO
Riverside
CAMDEN FARM
NEW WATER WORKS
Belpre
OLD PUMPING STATION
WEST VIRGINIA
OHIO
PARKERSBURG
Berry Run
Worthington
Little Kanawha R.
Marrtown
Newport

FIG. 1.

consideration was given to the matter in July, 1909, by Mr. Knowles, who called in Mr. F. G. Clapp, a geologist, as advisor on this subject. Mr. Clapp favored the well-water supply, but was also aware of the possibility of having to build new wells in future years, thus making it wise to secure at the outset plenty of land in the vicinity of the projected plant; such additional land would also protect the purity of the supply by avoiding surface pollution near the wells.

There was still one member of the Water-Works Commission who was unable to agree with the otherwise unanimous opinion as to the adequacy of the proposed well supply and its suitability as to quality. To obtain further data, James H. Fuertes and George W. Fuller, Members, Am. Soc. C. E., were engaged in November, 1909, to investigate and report on the proposed well supply, with a view to settling the differences of opinion between the city officials and the citizen members of the Water-Works Commission. They investigated the well-water project with considerable thoroughness, and found that, though it would be possible to obtain a 4 000 000-gal. supply from the gravel and sand layers underlying the bottom-lands in the neighborhood of the Camden Farm, it would be necessary that such a well system should extend along the river for at least 4 000 ft., thus making its cost greater than that of a filtered-water supply from the river. They recommended the adoption of the latter.

The city officials passed an ordinance authorizing the Board of Affairs to build the filtration works and appurtenances, but this authorization was nullified by an injunction suit which stopped the further expenditure of funds, made available by the bond issue for the purpose, without the approval of the Water-Works Commission. In the meantime Mr. L. E. Smith interested some of the leading citizens in a system of water supply consisting of a manifold of pipes laid in a sand bar in the Ohio River. Samuel M. Gray, M. Am. Soc. C. E., was then engaged to report on the advisability of adopting the so-called Smith system. His report, delivered on May 26th, 1910, being favorable, the works were built under his direction, and for 5 years or more the water has been clear, clean, and of satisfactory sanitary quality. Not only has the new water supply been satisfactory to the citizens, but it is gratifying to note that the new pumping equipment has effected an economy sufficient to pay the capital charges on the bond issue required by the new plant.

It is believed that the development of the new supply has placed on record many data of general interest to the Profession with regard to ground-water; and that this is so with respect to the investigation of the well-water supply by Messrs. Knowles, Fuertes, and Fuller, and in particular in the case of the somewhat novel system built by Mr. Smith under the direction of Mr. Gray. In placing before the Society the principal features of the Parkersburg water project, the writer feels of necessity bound to make liberal use of the reports of the above-mentioned gentlemen, and wishes to state that much of the following is an abstract compiled from their several reports in the preparation of which he is indebted to them.

## Geological Conditions.

The geological conditions of the Ohio Valley in the vicinity of Parkersburg were investigated at length by Mr. Knowles with the aid of Mr. T. G. Clapp.

Attention is first directed to the cross-sectional profile of the Ohio River at Lock and Dam No. 18 (Fig. 2), furnished by the writer from investigations made under his direction.

The bottom-lands on the West Virginia bank of the Ohio River, for a distance of 5 or 6 miles north of and above Parkersburg, contain more or less water-bearing strata. This is shown by numerous wells in successful use for domestic and manufacturing purposes within that tract, and is also confirmed by an examination of the materials removed in drilling these wells.

From a study of the history of the Ohio River in this immediate locality, it appears that the river bed has shifted its position materially along the part south of the promontory at Briscoe, where the rock of the West Virginia shore extends down to and beneath the river bed. At present the Ohio lies well to the west in the river bottoms, in some places nearly reaching the foot of the hills forming the rugged upland of the Ohio shore. In earlier days the river unquestionably skirted the West Virginia foot-hills. The result of this shifting has been the deposition of a substantial quantity of gravel and sand in the present bottom-lands, with a stratum of sandy gravel extending north from above the upper limits of the City of Parkersburg for about 5 miles, and having a varying width averaging perhaps $\frac{3}{4}$ mile.

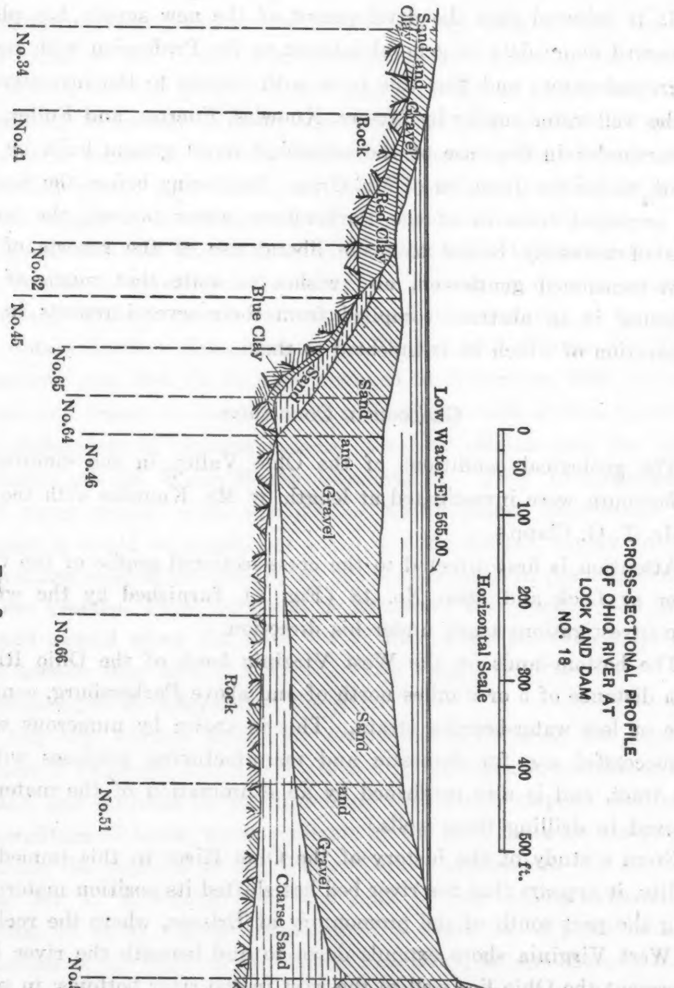

CROSS-SECTIONAL PROFILE
OF OHIO RIVER AT
LOCK AND DAM
NO. 18
0 50 100 200 300 400 500 ft.
Horizontal Scale
Elev. above sea level
590 ft.
580
570
560
550
540
530
Low Water El. 565.00
Sand and Gravel
Clay
Rock
Red Clay
Blue Clay
Sand
Sand and Gravel
Sand and Gravel
Coarse Sand
Rock
Blue Clay
Loam
No.34
No.41
No.62
No.45
No.65
No.64
No.46
No.66
No.51
No.50
No.1

FIG. 2.

This stratum varies in depth from perhaps 15 to 25 ft., and averages approximately 20 ft., according to available information. It is overlaid with a stratum of loam ranging in depth from 25 to 40 ft. The bottom of the sand and gravel stratum varies in elevation roughly from about 540 to 550 ft. above sea level, and its top surface from about 560 to 570 ft.

A more detailed statement of the local geology, with comments on its significance with reference to the ground-water supply of Parkersburg, is set forth with various comments by Mr. Clapp in the following paragraphs.

*Local Geological Details.*—In order to make sure of the adequacy of the present and future supply, the formation of the valley and its sediments was carefully considered. The real bottom of the Ohio Valley is not the present surface of the river or of the first and second bottoms, but is the bed-rock surface, or surface of the solid rock (sandstone and shale) underlying these. Along the part of the valley considered, this bed-rock surface lies from 50 to 125 ft. or more below the first bottoms, as determined by the depth of numerous wells and test borings.

The origin of the buried bed-rock surface is due to erosion at a time—thousands of years ago—before the overlying gravel filling existed, when the whole country stood higher above the ocean level, and hence the present bed-rock was at the surface of the ground. The deeper parts of the bed-rock valley (*i. e.*, those now buried more than 100 ft. below the present bottoms) were eroded by the ancient Ohio River as it cut its way deeper into the surface of the country while it was comparatively high, forming a sort of gorge below the general level of the bed-rock surface in the valley. Thus there were (and still are) three rock levels: (1) that of the gorge, now from 100 to 125 ft. below the present first bottoms; (2) that of the ordinary bed-rock in the valley, now from 50 to 90 ft. below the first bottoms; and (3) that of the surrounding hills, the tops of which stand from 200 to 300 ft. above the bottoms. The sand, gravel, and clay filling which forms the bottoms and higher unconsolidated terraces along the valley was formed at a more recent date than the rock valleys, due to various incidents in connection with the Ice Age, during which the northern part of the territory now occupied by the State of Ohio

was covered with a great glacier, damming the river valleys at various places.

For the most part, the buried bed-rock surface of the Ohio Valley is rather flat, and, under the Camden Farm and elsewhere, it lies from 50 to 90 ft. below the lower bottoms. On this rock bench rest the water-bearing gravels, from 20 to 30 ft. thick, from which it is proposed to draw the water supply by pumping from wells. The gravels are covered by clay, fine sand, and loam, reaching to the surface of the bottom-land. Many household wells driven into the gravels, and also geological inferences, have shown that the formation is similar and continues for miles up and down the main valley.

No part of the rock gorge underlies the Camden Farm, but at this place it is below the present river. Presumably it meanders from side to side of the valley, and farther up and down stream it may underlie the gravels, as it does at the test wells on the land of the Parkersburg Iron and Steel Company. In order to show that this deeper gorge will not interfere with the supply of the wells by draining off the water at a lower level, it may be stated that the deeper gorge is filled for the most part with finer material, such as fine sand and clay, and these do not hold large volumes of water. Moreover, the portion of the underflow of the river which follows the gorge is separated from the water in the upper gravels by layers of finer, more impervious material, which tend to keep the two bodies of water separate.

Another geological feature which needs consideration is the terrace known as the "sand plains region", lying back of Boreman Hill, and standing at an average elevation of 80 ft. above the first bottoms, or 130 ft. above the underlying bed-rock. The sand plains are about ½ mile wide, and extend from the bottom-lands of the Ohio Valley at Beechwood for ½ mile to Worthington Creek. It is a geological fact, based on good evidence, derived from what is known of similar sand plain regions at various places along the Ohio, that this strip was the ancient valley of Worthington Creek, or the Little Kanawha River. The depth of the sand and gravel filling there is unknown, but it is considerable; and through shallow wells which have been drilled there, it is known that it contains a large volume of water. This water is supplied by the rain which falls on the sand plains and flows westward, through the underlying sand and gravel, to the Ohio Valley,

where a large part of it passes through the gravels underlying the Camden Farm.

Thus there will be two sources of supply for the wells: the ground-water from the sand plain district, and that from the bottom-lands in the main valley of the Ohio, both bodies of water moving toward the wells. At no other locality in the vicinity could this double supply be obtained in such great quantity.

*Gravel-Water Versus Rock-Water.*—The question arises: Can a better or more adequate supply of ground-water be obtained by drilling to some lower level than is planned, that is, into the bed-rock? To this question the reply is decisively "no". The rocks underlying the river valleys and the entire vicinity of Parkersburg are similar in character to those in the surrounding hills, and consist of shales and sandstones of the Dunkard formation (upper barren coal measures), which contain very little water, and nowhere in these rocks could a sufficient supply be obtained. In fact, a supply as large as is necessary does not seem to exist anywhere in the shales and sandstones of the coal measures, or in the thin limestones scattered through them. Many hundred feet below the surface of the valley, a large enough supply could be found in limestones, but, even if it was practicable to sink wells to that depth, the water would be found to be salty and unfit for use.

*Estimated Yield of the Wells.*—Calculations by Mr. Clapp agreed with those by Mr. Knowles, that fourteen wells will yield from 3 000 000 to 4 000 000 gal. of water per day, which is more than is at present needed, and will suffice for some years to come. However, after from 8 to 12 years, it may be necessary to add a new well every year or two for several years, in order to keep pace with the increase in population and to replace possible deteriorating wells. Hence, plenty of land should be secured in the vicinity of the wells. Owing to the character of the water, and the known type of river gravels, the wells may last 15 years or more without the necessity of being replaced. In order that the supply may be maintained at approximately its initial quantity, the wells should be cleaned occasionally, and, if pumped with an air-lift, the air pressure should be reversed frequently, in order to blow out any fine material which may collect in the pores of the gravel close to the wells or in the strainers.

*Arrangement of the Wells.*—The water in the gravels is moving in two directions. A part of it, derived from the rainfall on the bottom-lands farther up the valley, is moving slowly down stream, approximately parallel to the Ohio River. A second part of the gravel-water, derived from the rainfall on the sand plains, is moving from them toward the river, in general at right angles to it. Therefore, the proposed arrangement of the wells in two lines, one parallel to the river and the other at right angles thereto, is the best for securing all available water. The wells should be at least 300 ft. apart.

## Hydraulics of the Local Well-Water Field.

So much discussion having arisen in Parkersburg as to the relative merits of a well-water supply and a filtered-water supply from the Ohio River when Messrs. Fuertes and Fuller made their examination in the fall of 1909, it was determined to conduct pumping tests for a period of at least 3 weeks. Accordingly, a deep-well pump was set up in one of the 12-in. wells sunk by Mr. Knowles. This pump was operated under the observation of James R. McClintock, M. Am. Soc. C. E., from January 18th to February 8th, 1910, and during this period quantities ranging from 480 000 to 575 000 gal. daily were pumped. Eight special observation wells were driven, and frequent studies were made of the water level therein, and also in thirty-one existing driven wells within 3 or 4 miles up the river on the Camden Farm site, including two wells in the sand plains east of the bottom-lands.

Water levels in the river were taken several times a day, and samples of the well water were collected frequently for analysis. The temperature of the well water was uniformly between 51 and 52° Fahr., and that of the river water during the period under observation ranged from 34 to 38° Fahr.

## Summary of Results of Studies of Ground-Water Supply, as to Source, Quality, and Quantity.

The results of the pumping tests are given in a report by Messrs. Fuertes and Fuller, dated February 21st, 1910, with perhaps unusual clearness, and a summary of this information is given at some length, as follows.

*Water-bearing Strata.*—The first essential in securing a ground-water supply was to ascertain the conditions with respect to the water-bearing strata, as it is obvious that without such strata a ground-water supply for the city was out of the question. Reference has already been made to the local geological conditions, and these may be further spoken of from the standpoint of water supply as to the character of the water-bearing strata.

The elevation of the Ohio River opposite the Camden Farm ranges from about 562 at low water to about 621 for extreme high water, which floods the bottom-lands to a depth of from 10 to 20 ft. At low water the depth of the river varies somewhat, but approximates 5 ft., according to the surveys made by the United States Government.

The present river bed opposite the low lands in question is made up largely of sand and gravel. The proportion of gravel and the range in gravel size vary considerably. The proportion of sand apparently increases with the distance from the natural dam at Briscoe.

Excellent building sand for local construction work is secured from the river bed by dredging at a point opposite the Camden Farm. Samples of this sand, after removing the gravel by screening, show what would be called a medium river sand, having an "effective size" of about 0.28 mm. Test wells sunk by Mr. Knowles, and from which samples of material were placed on exhibition in the rooms of the Water Commission, show a substantial proportion of relatively fine or medium-sized gravel from which the sand was largely eliminated in the process of removal from the ground. So far as it has been possible to ascertain, these bottom-lands, at a depth of about 40 ft. below the surface, contain a sand and gravel layer with the sand predominating. The sand is of medium size, filling the voids of the gravel and thus controlling the resistance of the stratum to the flow of water. The average depth of the layer is about 20 ft., and not more than one-fourth of it is above extreme low water. In a rough way, it appears that during dry years the water in the river is below the level of the surface of the sand and gravel stratum for about 4 months, and sometimes it is 3 or 4 ft. below for weeks at a time.

Deep wells show that, beneath this water-bearing stratum of sand and gravel, there are layers of shale and clay, rather irregularly spaced, and interspersed with more or less sand and gravel. So far as it has been possible to ascertain, however, the sand and gravel layers below

the main deposit are too thin and too variable as to continuity to make them worth serious consideration in connection with a municipal water supply.

The general conditions as to the present river bed at Lock and Dam 18, about 2 miles below Briscoe, are shown on the sectional profile, Fig. 2.

*Source of Ground-Water.*—The second step in studying a ground-water project was to ascertain the source of the water found in the water-bearing strata, and particularly the source and direction of the flow of water in such strata in the event that a municipal supply were to be taken regularly therefrom.

Inspection of local conditions shows at once that the promontory at Briscoe, with the rock extending across the river a short distance beneath the river bed, clearly precludes the possibility of any substantial quantity of ground-water coming from points above Briscoe and passing longitudinally down the bottom-lands. Accordingly, a water entering the sand and gravel stratum must come either from the river, by infiltration through the bottom and sides of the river-bed, or from the rainfall on the bottom-lands themselves or the upland district immediately tributary thereto.

Inspection shows that these bottom-lands are traversed longitudinally by Pond Run for the greater portion of the distance, and at the upper or northern end there is another run which passes diagonally across the bottom-lands and enters the Ohio at Vienna. These surface streams have a drainage area of some 5 sq. miles back in the hills. The hilly sections contain much clay and loam, and are generally of an impervious character, as distinguished from the so-called sand plains. Pond Run apparently has an almost impervious bed. Its slope is exceedingly light, and the fine materials deposited show that except at times perhaps of heavy rainfall, no appreciable quantity of water passes from this run into the water-bearing strata below.

Along the edge of the upland area there are a number of flowing wells and wet places, showing clearly that the water entering the ground in the hilly upland district meets impervious strata as it reaches the edge of the bottom-lands, and that it is easier for it to force its way to the surface than to enter the sand and gravel strata.

Of course, some rainfall on the bottom-lands themselves would reach the underlying strata of sand and gravel, but the fine and impervious

nature of the 40-ft. layer of loam precludes this from reaching any substantial volume. It must also be borne in mind that the loam stratum is flooded with muddy Ohio River water from time to time, and the surface at such periods is covered with a deposit of fine silt and clay.

During wet weather, perhaps when the water in the Ohio is high, it would appear that the water contained in the pores of the extensive sand and gravel layers is largely from the river, but to some extent it is from upland sources, the high river stage damming up some of the upland water.

Messrs. Fuertes and Fuller were firmly of the opinion that, during low-water stages in the Ohio, the water which can be drawn from the strata is that which has been stored within the pores of the material at times of previous high water. They were also certain that, during low stages in the river, as this stored water is removed or naturally flows away, whatever water enters these sub-surface strata of sand and gravel is largely of river origin. They concluded, therefore, that, as a source of supply for a municipality, the water from such strata, at the end of protracted droughts, must come from the Ohio through the bottom or sides of the present river bed along the bottom-lands below Briscoe.

*Disposition of Ground-Water.*—The next point on which it was necessary to make inquiry, in studying the feasibility of a ground-water supply, was the answer to the question: Where does the water entering the porous sand and gravel layers in question now go? The answer is shown clearly by the studies which have been made and the profiles of the water levels secured from different wells prior to the occurrence of high water during the winter of 1909-10, as well as in the study of various water levels or contours during and subsequent to the January rise in the Ohio. Unquestionably, the water in the porous sand and gravel layers, found at some distance below the bottom-land, ultimately makes its way more or less irregularly and slowly into the Ohio.

Several features of interest and importance were disclosed in studying the question of the relation between the passage of the water from the Ohio into what is practically a large underground storage reservoir, as above described, the water storage therein, and

also the passage of water therefrom back into the river. This is indicated by the diagrams, Figs. 3 to 10.

In the first place, it was found that the water in the porous sand and gravel layers did not rise to such high levels as those found in the river itself. This is indicated by the readings in test wells almost at the bank of the river; and shows that, even during fairly high velocities in the river, there is still so much mud and silt deposited on its sides and bottom that a considerable head is required to overcome the friction sufficiently to allow substantial quantities of the water to pass through this thin and more or less impervious deposit.

In the second place, in discussing the question of the disposition of the water stored in these sand and gravel layers, it is necessary to consider the character of the material, and especially the finer particles of sand, making up the water-bearing stratum or underground storage reservoir. In this regard the observations showed that considerable head was required in order to cause the water to flow in substantial quantities through the sand interspersed with gravel. For instance, it was found that the river water scarcely affected the level of the water in wells 2 000 ft. from the river, notwithstanding that in the river itself there was a 20-ft. rise during the period in which the observations were made.

This shows two important features: one is that this underground reservoir fills from the river only to a slight extent during floods of short duration; the other is that considerable water cannot be removed at a single point without creating quite a steep slope on which the water will flow to the point of removal. This means that the water stored in this underground reservoir can by no means be removed entirely; in fact, in order to remove a substantial portion, it would be necessary to establish points of withdrawal at quite frequent intervals throughout the area in question.

Pertaining, however, to the answer to the main proposition set forth under this heading, it may be said that the water stored in this sand and gravel layer during floods makes its exit into a falling river with quite steep slopes when the quantity stored is great. Considering these underground sand and gravel layers as a storage reservoir, therefore, it is seen that, after having once been filled during a protracted flood, the reservoir, in the absence of any pumping, slowly becomes emptied as the river falls. Here it may be pointed out that, although

the mud deposits on the bottom and sides of the river bed are of much significance in controlling the rate at which this underground reservoir will fill, they seem to be of less significance as regards its emptying, on account of the lifting of the deposits from the sides of the stream bed by the pressure of the ground-water in its flow to the river.

*Collection of Ground-Water.*—The next step in considering a ground-water proposition, particularly one like that at Parkersburg, where a flowing underground stream does not exist, is to ascertain the best means of drawing or collecting the water from the porous sand and gravel layers containing it.

It is true that during the pumping tests the bottom-lands were so saturated with water that the withdrawal of 500 000 gal. or more per day from a single 12-in. well made very little impression on the surrounding water levels. A cone of depression was noticeable, particularly during the latter portion of the pumping period, but the information obtained therefrom was slight when compared with that available from other sources. This is indicated by the diagrams, Figs. 3 to 10.

The studies by Messrs. Fuertes and Fuller showed clearly that during the pumping period, from January 17th to February 5th, 1910, the ground-water contours were nearly parallel to the river. In other words, there was substantially no difference in the elevation of the ground-water at approximately equal distances back from the river throughout the entire length of the bottom-lands. This fact tends to show that the quality and porosity of the underlying porous strata of these lands are not materially different throughout their length. It also shows that there is no substantial flow of water longitudinally through the porous strata forming the underground reservoir in question. Therefore, they concluded that it would not be judicious to provide for the removal of ground-water by a line of wells at right angles to the river bank. This is shown by Figs. 3 to 10.

The records obtained by Mr. Knowles in December, 1908, show that, even when less than 350 000 gal. per 24 hours were pumped from a 12-in. well, the water level was lowered in the well about 13 ft. without lowering the surface materially at points less than 200 ft. from the well. Furthermore, this cone of depression had its apex below the surface of the rock on which the sand and gravel stratum rests.

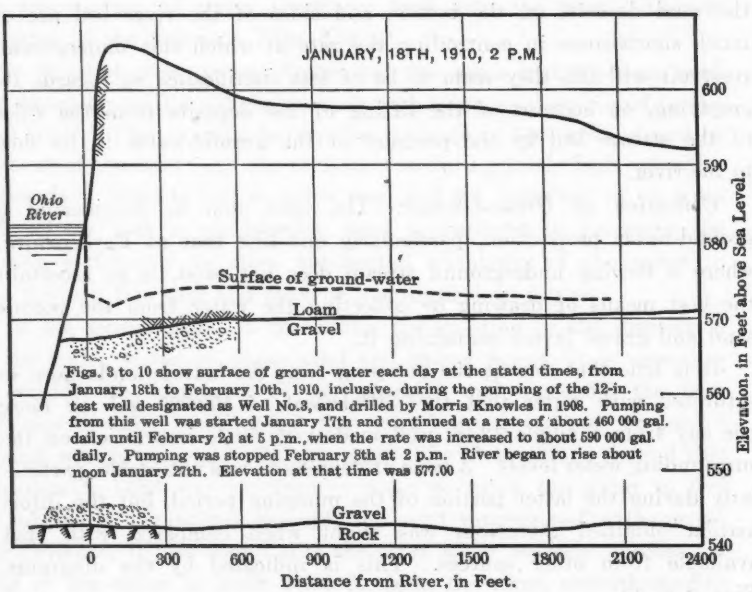

JANUARY, 18TH, 1910, 2 P.M.
Ohio River
Surface of ground-water
Loam
Gravel
Figs. 3 to 10 show surface of ground-water each day at the stated times, from January 18th to February 10th, 1910, inclusive, during the pumping of the 12-in. test well designated as Well No.3, and drilled by Morris Knowles in 1908. Pumping from this well was started January 17th and continued at a rate of about 460 000 gal. daily until February 2d at 5 p.m., when the rate was increased to about 590 000 gal. daily. Pumping was stopped February 8th at 2 p.m. River began to rise about noon January 27th. Elevation at that time was 577.00
Gravel
Rock
600
590
580
570
560
550
540
0
300
600
900
1200
1500
1800
2100
2400
Elevation, in Feet above Sea Level.
Distance from River, in Feet.

FIG. 3.

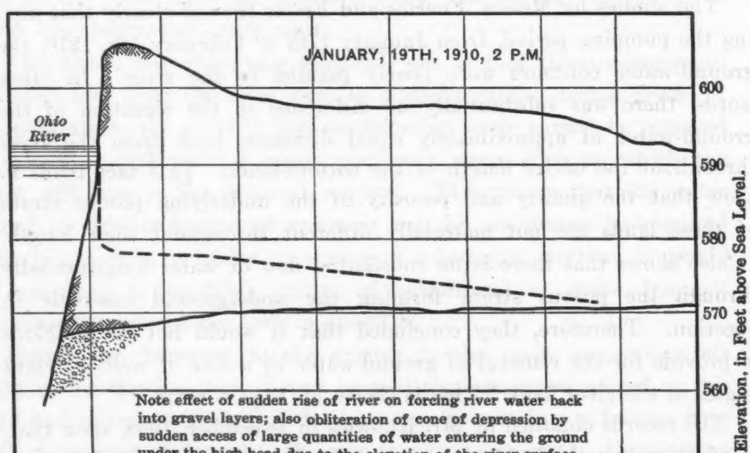

JANUARY, 19TH, 1910, 2 P.M.
Ohio River
Note effect of sudden rise of river on forcing river water back into gravel layers; also obliteration of cone of depression by sudden access of large quantities of water entering the ground under the high head due to the elevation of the river surface.
600
590
580
570
560
550
540
0
300
600
900
1200
1500
1800
2100
2400
Elevation, in Feet above Sea Level.
Distance from River, in Feet.

FIG. 4.

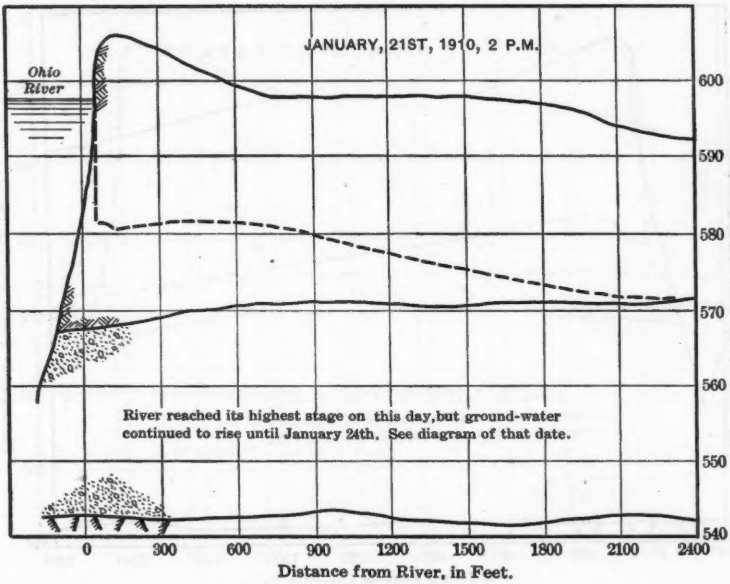

JANUARY, 21ST, 1910, 2 P.M.
Ohio River
River reached its highest stage on this day, but ground-water continued to rise until January 24th. See diagram of that date.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 5.

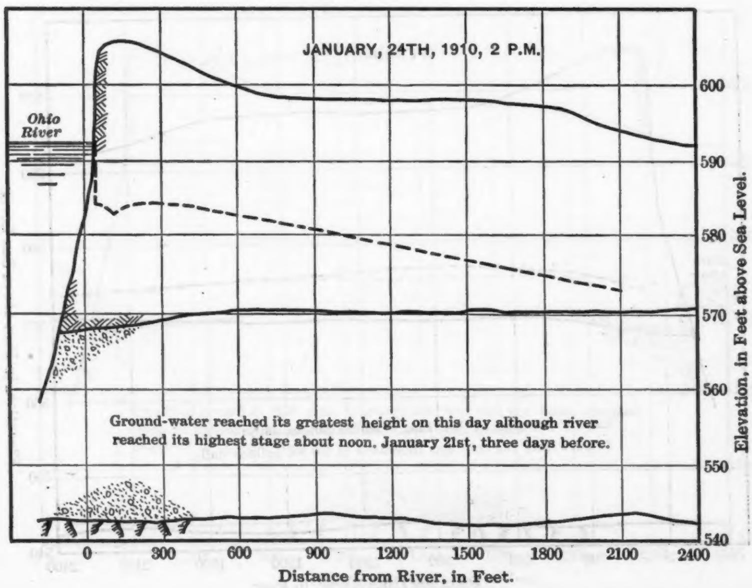

JANUARY, 24TH, 1910, 2 P.M.
Ohio River
Ground-water reached its greatest height on this day although river reached its highest stage about noon, January 21st, three days before.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 6.

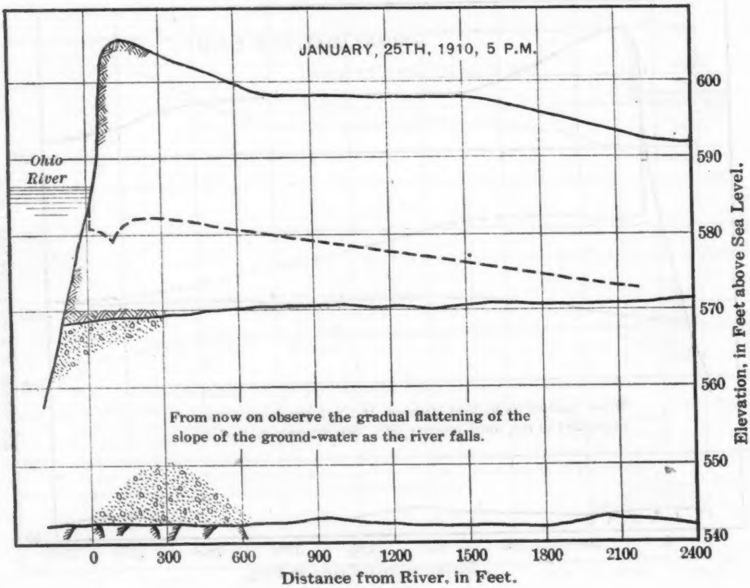

JANUARY, 25TH, 1910, 5 P.M.
Ohio River
From now on observe the gradual flattening of the slope of the ground-water as the river falls.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 7.

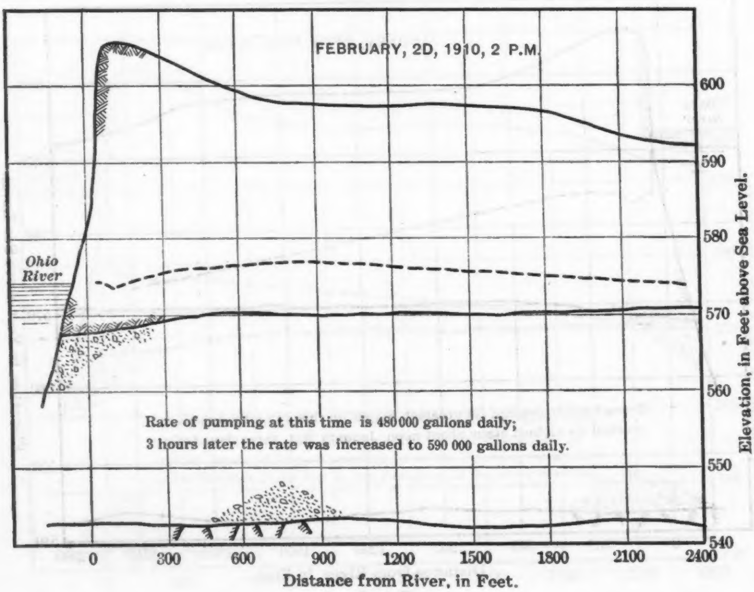

FEBRUARY, 2D, 1910, 2 P.M.
Ohio River
Rate of pumping at this time is 480 000 gallons daily;
3 hours later the rate was increased to 590 000 gallons daily.
600
590
580
570
560
550
540
Elevation, in Feet above Sea Level.
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.

FIG. 8.

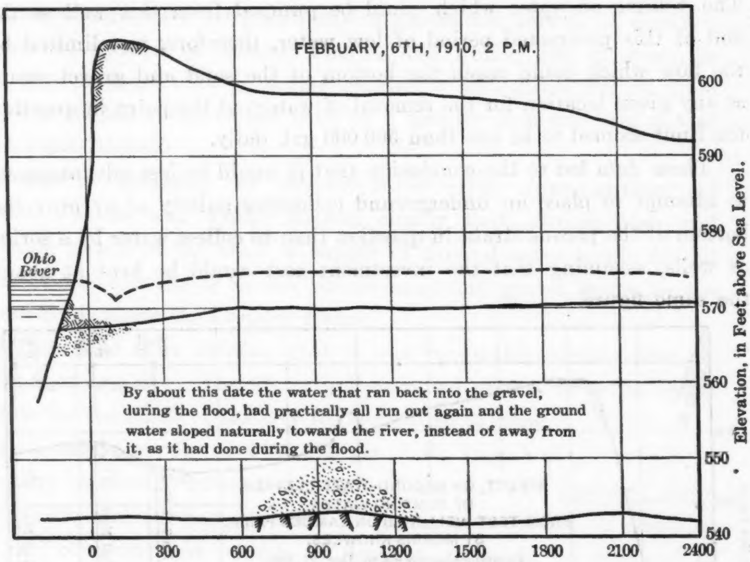

FEBRUARY, 6TH, 1910, 2 P.M.
Ohio River
By about this date the water that ran back into the gravel, during the flood, had practically all run out again and the ground water sloped naturally towards the river, instead of away from it, as it had done during the flood.
600
590
580
570
560
550
540
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.
Elevation, in Feet above Sea Level.

FIG. 9.

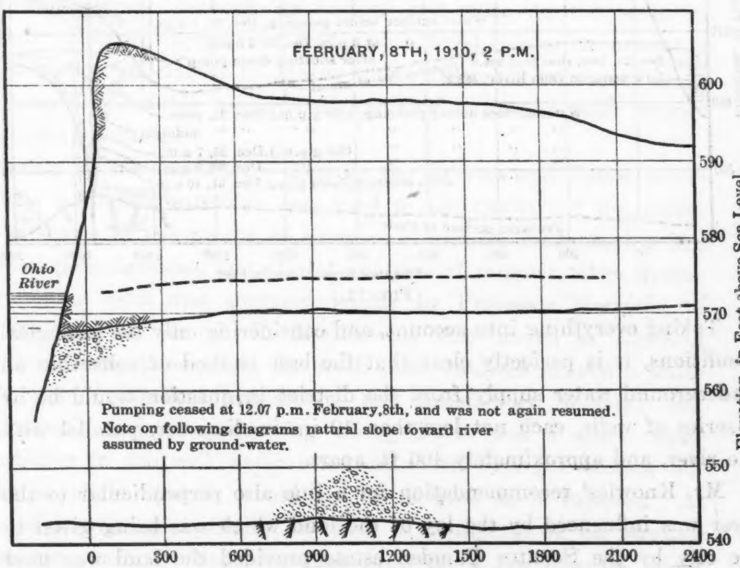

FEBRUARY, 8TH, 1910, 2 P.M.
Ohio River
Pumping ceased at 12.07 p.m. February, 8th, and was not again resumed. Note on following diagrams natural slope toward river assumed by ground-water.
600
590
580
570
560
550
540
0
300
600
900
1200
1500
1800
2100
2400
Distance from River, in Feet.
Elevation, in Feet above Sea Level.

FIG. 10.

The volume of water which could be pumped from this well at the end of this protracted period of low water, therefore, was limited by the flow which could reach the bottom of the sand and gravel strata at any given location for the removal of water; at the point in question the limit seemed to be less than 500 000 gal. daily.

These data led to the conclusion that it would be less advantageous to attempt to place an underground collecting gallery at or near the bottom of the porous strata in question than to collect water by a series of wells, assuming that the investment cost would be kept at about the same figure.

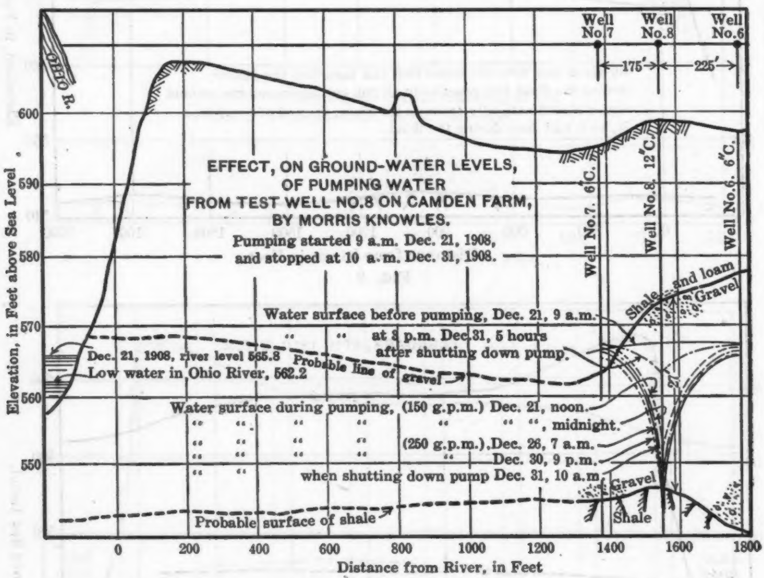



FIG. 11.

Taking everything into account, and considering only the geological conditions, it is perfectly clear that the best method of collecting an underground water supply from the district in question would be by a series of wells, each not less than 10 in. in diameter, parallel with the river, and approximately 400 ft. apart.

Mr. Knowles' recommendation for a line also perpendicular to the river was influenced by the lay of the land which was being given to the city by the Senator Camden estate provided the land was used for water-works purposes.

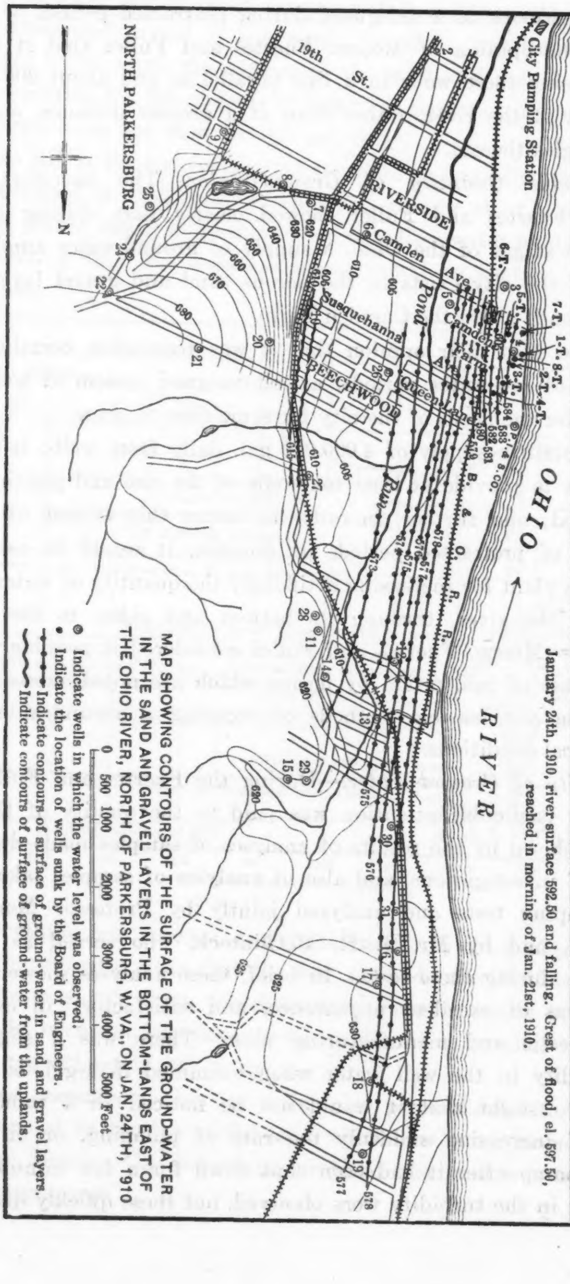
Furthermore, as a safeguard during protracted periods of drought, it was the opinion of Messrs. Fuertes and Fuller that it would be wise to sink these wells in a line parallel to and about 200 ft. from the bank of the river, rather than at a greater distance, and not at right angles thereto.

*Available Quantity of Ground-Water.*—The investigations of Messrs. Fuertes and Fuller showed clearly that, during high and moderate stages of the Ohio, a supply of ground-water ample for all needs of the city exists in the porous sand and gravel layers in the bottom-lands above the Camden Farm.

It was also their opinion that it was reasonably certain that, at the end of low-water periods, a well-designed system of wells would provide for the needs of the city for some time to come.

To obtain a supply of 4 000 000 gal. daily from wells, it would be necessary to provide at least ten wells of the size and positions above mentioned; and further, in order to insure this volume of water at the end of protracted periods of drought, it would be essential to provide a plant for increasing artificially the quantity of water filtering in from the river, through its bottom and sides, to the wells in question. Messrs. Fuertes and Fuller considered it possible to secure an increase of infiltration by means which are noted subsequently in describing the feasible methods of securing a ground-water supply under local conditions.

*Quality of Ground-Water.*—During the Fuertes and Fuller investigations particular attention was paid to the quality of the water, both as shown by the results of analyses of samples made during Mr. Knowles' investigations, and also of analyses of samples taken during the pumping tests and analyzed jointly by Professor Merriam, of Marietta, and by Mr. J. R. McClintock, who served as Resident Engineer during these tests. In brief, these analyses showed a water which was of excellent appearance, and satisfactory in every way for domestic and manufacturing uses. There was a slight tinge of turbidity in the well water when examined in 1-gal. bottles, but it was so slight that it would not be noticed in a tumbler. At times of increasing suddenly the rate of pumping, or on starting up the pump after it had been shut down for a few minutes, slight increases in the turbidity were observed, but these quickly disappeared




MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM-LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W.VA., ON JAN. 24TH, 1910

◎ Indicate wells in which the water level was observed.
• Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.

FIG. 12.

All samples analyzed were tested without any effort to eliminate the slight turbidity mentioned.

Briefly stated, the bacteria in the well water ranged from 14 to 170, and averaged 63 per cu. cm. The total hardness of the well water ranged from 76 to 98, and averaged 90 parts per million. The river water samples collected on the same days showed a total hardness ranging from 52 to 71, and averaged 63 parts per million. Corresponding figures for wells in the Beechwood district were 88, 102, and 94, respectively.

Samples of well water taken from the test well were analyzed carefully for iron, and found to contain about 0.5 part per million. There was no sign of this iron precipitating out, and experience elsewhere shows that, with the quantity of iron above stated, difficulties are not encountered.

Samples of water taken from test wells contained rather objectionable quantities of iron in some instances, evidently due to the water dissolving iron during long intervals of standing in contact with the metal.

Slightly increased quantities of iron were found in the water of the well at the steel works, but not to an objectionable degree. This greater quantity of iron is explained partly by the flow of water through a long line of pipe and partly by the connection which the well has with underground porous strata where the water remains in contact with pervious layers for many months and perhaps years.

Messrs. Fuertes and Fuller, taking everything into consideration, had no hesitancy in stating that the quality of the water from the porous sand and gravel layers of the bottom-lands at and above the Camden Farm is thoroughly satisfactory for domestic and manufacturing purposes. They placed the water from such wells in the same general class as filtered water obtained from a properly built and well operated filter plant treating water from the Ohio River in the vicinity of Parkersburg.

There are some slight differences in the quality of the water from wells and from filters, the principal feature of which would be the hardness on account of the wells yielding water coming largely from the Ohio River at times when it is in flood and fairly soft. Messrs. Fuertes and Fuller did not attach much importance to this difference, because ultimately, during low-water periods, the well water would

also have its source in the Ohio. So far as quality is concerned, they put waters from filters and from wells in the vicinity of the Camden Farm on a parity. However, the writer is somewhat partial to the natural water when equally free from bacteria and other objectionable qualities.

### Most Practicable Method of Securing a Ground-Water Supply.

Messrs. Fuertes and Fuller discussed and considered from various viewpoints the most practicable method of securing a ground-water supply, and their conclusions may be briefly summed up as follows.

It is not feasible to secure a municipal water supply for the City of Parkersburg by galleries or wells running at right angles to the Ohio River.

Wells of very large diameter, or tunnels, would not be a fruitful investment, on account of the steep slopes of the cones of depression which would be formed around the points of withdrawal of water. Small wells, not less than 12 in. in diameter, afford by far the best way of collecting the ground-water supply from the porous sand and gravel layers in the bottom-lands at and above the Camden Farm.

These wells should be in a line parallel to the river, about 400 ft. apart, and about 200 ft. from the bank.

A deep tunnel for receiving the flow of the wells is far less expedient than a force main receiving the waters from individual wells, due to the expense of tunneling at the depths required, and other local conditions.

### Description of Well-Water System as Considered by Messrs. Fuertes and Fuller.

The studies of the plant necessary to carry out most economically and advantageously the methods just described of securing a ground-water supply from the locality in question, are briefly described as follows.

*Extent.*—Ten wells should be established, in a line extending along the river front, approximately 200 ft. from the bank, and 400 ft. apart.

*Location.*—As the well layout would cover about 4 000 ft., it is not feasible to place all the wells on the Camden Farm, or, in fact, all below the Steel Works, as the porous underground strata do not extend a sufficient distance down stream. It is assumed, therefore, that the well plant will extend to above the Parkersburg Iron and

Steel Works. Prudence, however, would suggest that they should not reach above the lower end of Neal's Island, on account of the shallowness of the channel east of that island, the absence of securing velocities therein, and the imperviousness of the stream bed.

*Wells.*—It is contemplated to sink shafts $7\frac{1}{2}$ ft. in diameter, brick lined, and 40 ft. deep, or down to the top of the porous sand and gravel layer. In each there should be a cased well, 12 in. in diameter with a brass strainer for a length of at least 20 ft. at the lower end. This 12-in. pipe would be connected with a centrifugal pump placed at the bottom of the $7\frac{1}{2}$-ft. shaft, the masonry wall of which would be extended up to above extreme high water, some 12 ft. above the ground level in this locality.

COMPARISON OF WELL-WATER SUPPLY WITH FILTERED SUPPLY FROM THE OHIO RIVER.

In the report of Messrs. Fuertes and Fuller comparative estimates of cost were given, both for the well-water supply and filtered river-water supply to provide an average daily yield of 4 000 000 gal. and be capable of supplying water for short periods at a maximum rate of 6 000 000 gal. per 24 hours.

In substance, this report stated, on the basis of figures detailed therein, that the cost of construction and of operation of a first-class well system and a first-class filter system at the Camden Farm site were substantially on a parity as to the quality of water and as to the cost, both of construction and of operation, and that either plant could be depended on to supply such quantity of water as would be needed by the City of Parkersburg for some years to come. Mention is to be made of the fact, with reference to the well-water supply, that the estimates of cost include a suitable charge for land required to insure the adequacy of the system.

Further, the report stated that, by placing the filter plant nearer the site of the present pumping station, the investment cost could be made less for the filter plant, but with no substantial difference in operating and maintenance expenses between either plant at either site. The report finally recommended that all interests of the city would be best served by the establishment of a filter plant, near the site of the then existing water-works pumping station, to supply the city with properly filtered Ohio River water.

SMITH INFILTRATION SYSTEM.

An injunction suit, as already stated, prevented the city officials from acting independently of the full Water-Works Commission in building the proposed filtration system, as above recommended. Then, at the request of various prominent citizens of Parkersburg, the Smith system came up for consideration.

In May, 1910, Mr. Samuel M. Gray, reported on water-works improvements with special reference to the so-called Smith strainer-pipe system of filtration, his conclusions in brief being as follows:

1.—This system, under local conditions, is both feasible and practicable for furnishing an ample supply of water of suitable quality for the present needs of the city.

2.—If properly built and operated, this system will furnish water of a better quality than a mechanical filter plant, as regards steam-raising purposes, on account of the slight increase in permanent hardness which is caused by the use of a coagulant.

3.—Though higher in first cost than a mechanical filter plant, the operating cost of the Smith system would be materially less, amply so, in fact, to warrant the higher first cost.

4.—That water from the proposed driven wells would be inferior in quality to that from the Smith system, and, further, that such wells would be very likely to lessen in yield in course of time, and the water grow harder and contain more iron, with also the possibility of local contamination from surface sources.

On considering this matter further, Mr. Gray modified the detailed plans by recommending that the infiltration system should be divided into five units, with a separate suction pipe for each unit. The advantage of this was to facilitate back-flushing of the sand overlying the respective units of pipe manifold systems, and for making repairs and renewals without affecting seriously the service to the city. This recommendation was authorized, and, for the sake of economy, it was decided to abandon the so-called Camden Farm site and build a new pumping station on the river bank nearest the site selected for the infiltration system in order to economize in the cost of suction pipe. The relative locations are shown on Fig. 1.

Mr. Gray suggested using a greater length of slightly smaller strainer tubes, in order to increase the strainer surface. He also pre-

pared the plans and specifications for the construction of the infiltration system, as well as for other improvements to the water-works plant.

Contracts were then let for the infiltration system, and the work progressed to completion under the direction of Mr. Gray and according to the plans and specifications prepared by him. Mr. W. G. Wheelock was Resident Engineer.

## Unusual Features of the Contract for the Smith Infiltration System.

As there had been considerable skepticism regarding the successful working of the Smith system, no similar works of comparable size having been in operation, the contract was made unusually exacting. Only 45% of the value of work done was paid on monthly estimates, the remainder being retained until 30 days after the test period of 364 days after the beginning of successful operation, as guaranteed. The contractor made the following guaranties for the system:

1.—The capacity to be 170 000 gal. per hour;

2.—The supply to be satisfactory, to the Water-Works Commission and the Board of Affairs, for domestic and industrial uses;

3.—The water to be of pleasing appearance, practically clear and colorless, and to contain not more than 5 parts per million of turbidity;

4.—With 3 000 bacteria per cu. cm. in the raw water (or higher), the filtered water to contain not more than 3% of that number; with less than 3 000 per cu. cm. in the raw water, the filtered water to contain not more than 100 per cu. cm.; counts to be on agar plates incubated at 20° cent., for 48 hours;

5.—Filtered water to contain on an average not more than 30 parts per million of total hardness in excess of Ohio River water;

6.—Iron in solution not to exceed 0.7 part per million;

7.—The foregoing requirements to be shown by daily tests covering a period of 364 days from the date of beginning operation.

In case of failure to comply with the guaranty, the city was under no obligation to pay the retained 55%, and the completed plant was to become the property of the city.

The contractor was required to maintain the plant in good condition and thorough repair until acceptance.

CONSTRUCTION DETAILS OF THE SMITH INFILTRATION SYSTEM.

Under the contract, three units were completed and put in service on November 12th, 1911. The other two units were completed and put in service on December 15th, 1912. The plant has been in continuous operation ever since, except for 4 days during a flood which broke the former highest record by 5 ft., and thereby partly submerged the boilers and engines.

Fig. 13 shows the arrangement of the pipes in the infiltration system, and illustrates the appearance of the units while being laid.

The site of the plant is on the lower end of the bar at the foot of Vienna Island, at a point where the natural top of the bar forming the bed of the river is about 1 ft. below the low-water level. By inquiry, and the examination of Government hydrographic maps made about 10 years previous to the preparation of the plans, it was concluded that this bar had been practically stable for many years.

The system of five units of filters was placed by using two cofferdams of the ordinary box type common on the Ohio River lock and dam construction; and the suction pipes to the power-house were laid in an open trench, with the excavated material banked on each side. The work was done during the low-water seasons of 1911 and 1912, with the top of the coffer from 7 to 8 ft. above low water.

At this point the fluctuation of the water is about 58 ft., and stages up to 40 ft. are of annual occurrence. Coffer No. 1, enclosing Units 1 to 3, was built and removed in 1911, and Coffer No. 2 in 1912.

In the unwatered coffer the excavation for the pipes was made to 5½ ft. below low water. On the floor thus formed a 6-in. layer of gravel was placed, on which the pipes were laid and covered with 12 in. of gravel; on the gravel a 3½-ft. layer of clean washed sand was laid, bringing the top of the sand to within about 6 in. of the low-water line, or a few inches higher than the natural surface of the bar.

The infiltration system of five units thus formed is 110 ft. wide and 656 ft. long (1.65 acre), and is 5 ft. deep. The gravel was the washed and screened Ohio River product which passed a 1½-in. mesh, and with the sand removed. The sand was taken from the Ohio

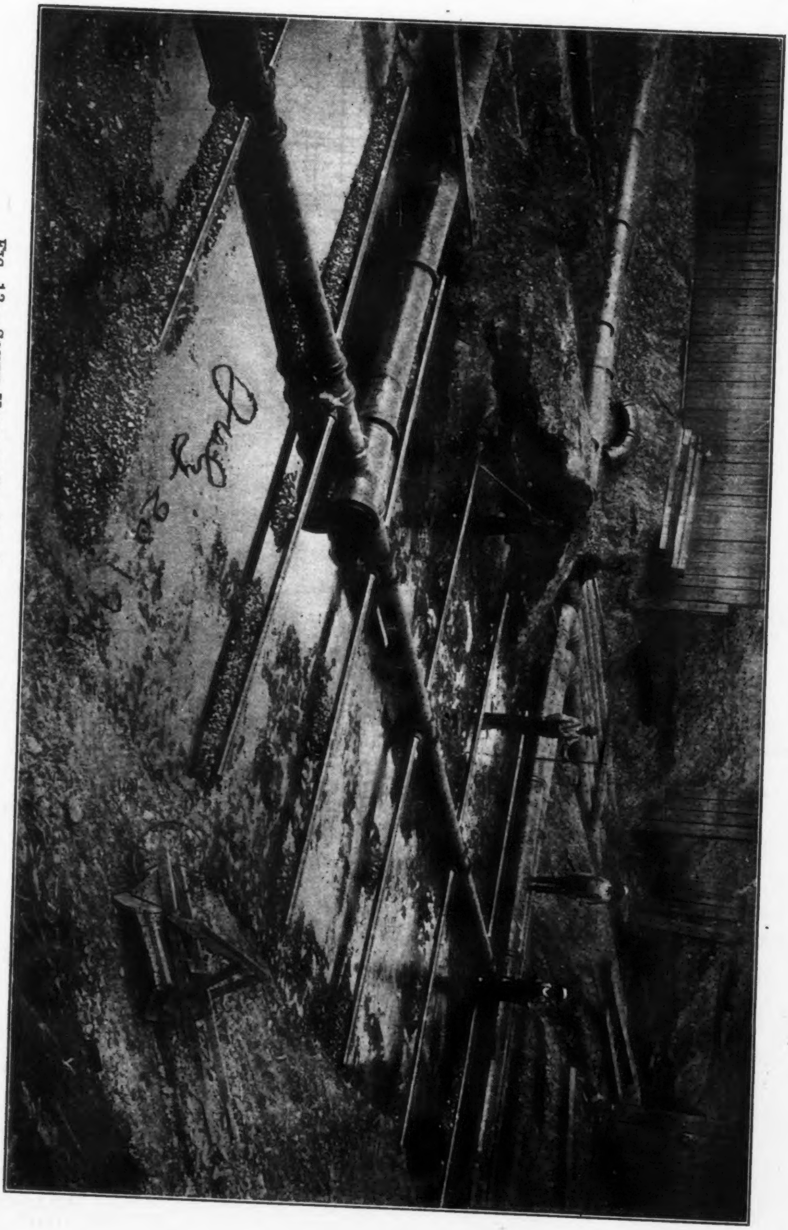


FIG. 13.—SOUTH HALF OF SECTION No. 3, PARKERSBURG, W. VA., WATER SUPPLY.

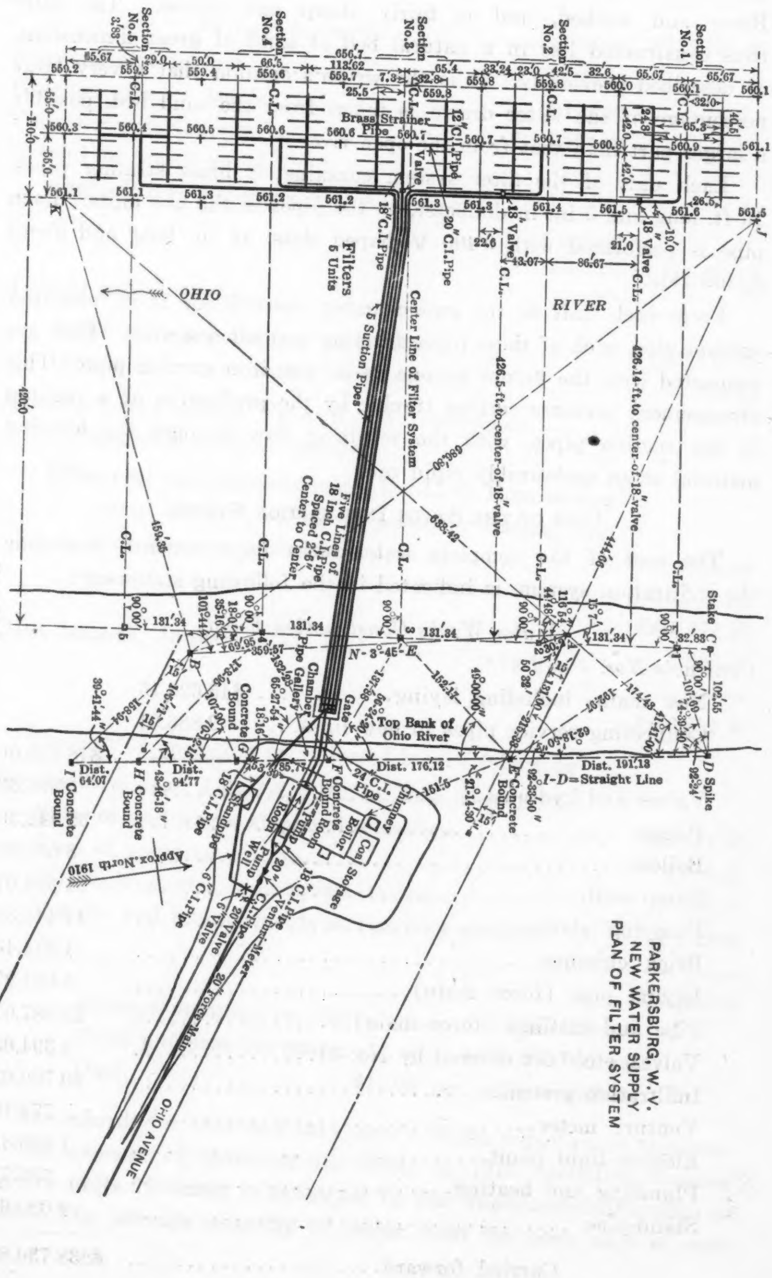

PARKERSBURG, W.V.
NEW WATER SUPPLY
PLAN OF FILTER SYSYEM
Section No.1
Section No.2
Section No.3
Section No.4
Section No.5
Brass Strainer Pipe
12" C.I. Pipe
18" C.I. Pipe
20" C.I. Pipe
18" Valve
Filters 5 Units
5 Suction Pipes
Center Line of Filter System
426.5 ft. to center of 18" valve
426.14 ft. to center of 18" valve
Five Lines of 18 inch C.I. Pipe Spaced 2'-6" Center to Center
OHIO
RIVER
Top Bank of Ohio River
Pipe Gallery
Gate Chamber
Concrete Bound
I-D = Straight Line
D Spike
Stake C
Dist. 176.12
Dist. 191.18
Dist. 64.07
Dist. 94.77
H Concrete Bound
I Concrete Bound
E Concrete Bound
F Concrete Bound
24" C.I. Pipe
Chimney
Boiler Room
Coal Storage
Pump Room
Pump Well
Standpipe
Venturi Meter
6" Valve
20" Valve
20" Force Main
OHIO AVENUE
Approx. North 1910

FIG. 14.

River and washed, and is fairly sharp and coarse. The filter thus constructed lies in a natural bed of sand of great dimensions, as described previously. It is the writer's opinion that a very large percentage of the water drawn is taken from the sand bed, possibly a larger part than that from the open river.

Each unit of the filter system contains 32 brass strainer pipes, 16 ft. long and 5 in. in diameter, or 160 pipes in the five units. Each pipe is perforated with 9 400 V-shaped slots, $1\frac{3}{8}$ in. long and about $\frac{1}{70}$ in. wide.

From each unit to the gate-chamber there is an 18-in. cast-iron suction pipe, each of these pipes forming a small reservoir. They are connected with the pumps by one 24-in. cast-iron suction pipe. This arrangement prevents serious trouble by the production of a vacuum in the suction pipes, with the resulting flow through the filtering material at an undesirably rapid rate.

### Cost of the Smith Infiltration System.

The cost of the complete water-works improvements, including the infiltration system, is indicated in the following statement:

*Water-Works Construction Fund.*

| | | |
|---|---|---|
| *Contracts Nos. 1 and 2:* | | |
| New mains including laying | $45 626.15 | |
| Connecting service pipes to new lines | 2 883.91 | |
| | | $48 510.06 |
| Valves and hydrants | | 5 936.53 |
| Pumps | | 32 442.30 |
| Boilers | | 8 055.30 |
| Pump well | | 21 556.67 |
| Pumping station | | 11 441.32 |
| Brick chimney | | 1 374.43 |
| Laying pipe (force main) | | 5 021.27 |
| Pipe and castings (force main) | | 15 987.01 |
| Valves, etc. (not covered by No. 3) | | 2 394.63 |
| Infiltration system | | 80 700.03 |
| Venturi meter | | 772.10 |
| Electric light plant | | 1 953.81 |
| Plumbing and heating | | 555.72 |
| Stand-pipe | | 2 029.65 |
| Carried forward | | $238 730.83 |

| | | |
|---|---|---|
| Brought forward | | $238 730.83 |
| Pumping station lot | $1 700.00 | |
| Expense, sale of bonds | 9 131.30 | |
| Tests for water supply | 2 435.20 | |
| Refund to water-works fund, on account of preliminary engineer's report, test wells, etc. | 5 000.00 | |
| | | 18 266.50 |
| *Amount paid to engineers:* | | |
| Preliminary reports and well tests | | 10 419.39 |
| Design and supervision of construction | | 11 801.92 |
| *Miscellaneous expenses, Water-Works Commission, Not chargeable to above items:* | | |
| Office rent and expenses, clerk, Dr. L. O. Rose testing water, electric wiring to pumping station, and miscellaneous expenses | | 4 952.58 |
| | | $284 171.22 |
| Cash balance, January 31st, 1914 | | 2 658.76 |
| | | $286 829.98 |

The cost of operation is indicated by the following:

For the fiscal year, July 1st, 1912, to June 30th, 1913:

| | |
|---|---|
| Salaries and wages at pumping station | $5 541.00 |
| Fuel at pumping station | 3 319.61 |
| Oil, packing, etc., at pumping station | 889.76 |
| Repairs and improvements at pumping station | 2 142.09 |
| | $11 892.46 |

Average pumpage per day........ 2 955 000 gal.

Total pumpage for year.......... 1 078 675 000 gal.

Cost.................... $11.03 per million gallons.

The records for 1914 and 1915 show a slight reduction in this cost. The expense of operating the filters in 1915, including flushing the five units 73 times, is estimated by the Superintendent's office as $673. The average quantity of water used for each unit at each

flushing is 40 000 gal. Although the tank capacity is 60 000 gal., it is the custom, in operation, to use 40 000 gal. at each flushing. The cleaning and flushing is practically all the expense which has been incurred in excess of pumping out of the open river. The time required to pass the 40 000 gal. through a filter is from 15 to 20 min. When pumping out of the open river with the old plant, some trouble was caused by ice, leaves, etc., but it is believed that the cost of pumping from the filters and keeping them cleaned is no greater than that of pumping from the open river.

The maximum pumped in one week in 1915 averaged 3 760 000 gal. per day and 154 166 gal. per hour. For 2 weeks after the great flood of 1913, the average pumpage and consumption exceeded 5 500 000 gal. per day.

The water has shown a little turbidity about three times, at river stages of about 45 ft. The writer has observed turbidity in that drawn from his service pipes only once.

From May 1st, 1913, to January 31st, 1914, daily tests of the water were made under the direction of the Commission by Dr. L. O. Rose in his laboratory at Parkersburg, and report thereof is shown in part in Appendix I. This report shows that the quality of the water produced by the filters conformed in every respect with the terms of the contract.

From time to time since making this series of daily tests, Dr. Rose has made tests of the water, and the data obtained by him for the calender year, 1915, are included in the Appendix. The water has been tested by numerous others, but the writer did not, until the summer of 1916, hear of a report indicating that it is not as good as required under the contract.

A recent report of it, prepared for the City of Wheeling, W. Va., has questioned its being of the highest standard. Since March, 1916, samples of water have been shipped monthly to the West Virginia State Bacteriologist, at Morgantown, W. Va., for test. Of about 14 shipments, several, when tested after standing a day or more, have been reported "unsafe". From February 15th to October 27th, 1916, Dr. Rose tested 38 samples and found *bacilli coli* present in 11, and absent in all the others. For each of these 11, 2 tests were made of 1 cu. cm. and 10 cu. cm. each. In 3 of the samples *coli* were present in both tests, and in the other 7, it was absent in the 1 cu. cm. Of the other 27

samples, the two tests were made for 19 of them and only the smaller quantity for the other 8.

On account of these reports, early in September, 1916, Mr. Gray was engaged to have all the filter beds examined and cleaned. He assigned G. H. Leland, M. Am. Soc. C. E., to carry out this work personally and Mr. Leland was engaged in doing this for nearly a week. The writer spent several hours with him on his second day of inspection. At several places there were holes in the sand and gravel covering. The holes were from 5 to 10 ft. in diameter, and were formed like craters. They extended down to within 1 or 2 ft. of the level of the brass strainer pipes. The work done under Mr. Leland's supervision consisted of stirring the deposit on top of the filter beds with a powerful force pump, at the same time giving each section numerous back-flushings, and thereafter filling the holes with fresh gravel and sand. In the 10 samples tested by Dr. Rose since that work was done, no *coli* were found. Of these 4 were tested in both 1 cu. cm. and 10 cu. cm., as previously explained, and the other 6 in only the smaller quantity.

The writer is of the opinion that tests should be made at regular intervals, weekly or more frequently; but, on account of the confidence in its quality, it appears that such an important work to safeguard the health of the community has not been done regularly.

Under date of June 10th, 1913, Mr. I. L. Birner, Bacteriologist, certified to Contractor Smith, after a series of tests in a laboratory at the pumping station, that:

"An examination of the results of 400 separate analyses of the filtered water at the Parkersburg filtration plant, made between January 23d and June 4th, 1913, covering a widely varying set of conditions, from a river stage at 5.8 to 58.9 ft., and with a river turbidity varying between the extreme limits of 15 and 1 950 parts per million, shows a clear water, with an average total hardness of 10.5 parts per million in excess of the river water; the total iron in solution, of the filtered water, was 0.19 of one part per million, against an average of 5 to 16 parts in the river, and a bacterial removal, or bacterial efficiency, of 98.28 per cent."

Until recent years an accurate count of typhoid cases in the city has not been available. However, the public impression is that, previous to 1912, the yearly number was comparatively very large.

For 1915 there were twenty-two cases which started in the city and ten cases which were brought in after illness commenced, making thirty-two cases in all. Of the twenty-two, three were traced to a well, three were railroad employees, and one was a United States Clerk, and these men were out of the city much of their time.

Dr. Stone, the City Health Officer in 1914, informed the writer that all the typhoid cases in the city that year except four were traced definitely to other causes than the city water.

Dr. Rose has stated that not one recent case of typhoid has been proved to have come from the city water supply, and this report is confirmed by Mr. Nearn, City Health Officer, whose present duty includes an investigation of the cause and history of all such cases. However, the monthly reports of typhoid cases in 1916 show a little higher numbers than for 1915.

Thus far, the system has been entirely satisfactory to the citizens. Its success has been fully equal to the most sanguine hopes of the writer when, as a commissioner, he voted for its adoption. However, he realizes that the experimental stage is not past.

Three units are now ending their fifth year of service, and the other two their fourth, with comparatively little indication of failure in quantity, or in the quality of the water.

However, the writer knows that it is only a question of years when the sand and gravel over the strainer pipes will have to be replaced by fresh clean gravel and sand; the principal question being the number of years before this will become necessary in order to maintain the good quality of the water. He is also impressed with the great importance of vigilance and care in its operation, and of systematic daily or weekly tests of the water.

# APPENDIX I

## Report of Tests of the Water from the Filtration Plant and the Ohio River, for January, 1914.

By Lonzo O. Rose, M. D., Parkersburg, W. Va.

The results are given in parts per million.

| | Date. | Turbidity. | Alkalinity. | Incrustants. | Total hardness. | Iron. | Bacteria per cubic centimeter. | Rate of pumping. | Vacuum. | Stage of river, in feet above low water. |
|---|---|---|---|---|---|---|---|---|---|---|
| R. | Jan. 1 | 70 | 14 | 55.0 | 69.0 | 3.26 | 600 | | | |
| F. | " 1 | Clear | 26 | 57.5 | 83.5 | 0.02 | 13 | 2 200 | 14.5 | 10.1 |
| R. | " 2 | 50 | 21 | 60.0 | 81.0 | 2.70 | 200 | | | |
| F. | " 2 | Clear | 37 | 60.0 | 97.0 | 0.01 | 4 | 2 200 | 17.0 | 9.2 |
| R. | " 3 | 50 | 16 | 60.0 | 76.0 | 2.30 | 300 | | | |
| F. | " 3 | Clear | 30 | 55.0 | 85.0 | 0.1 | 8 | 2 300 | 17.0 | 8.4 |
| R. | " 4 | 20 | 16 | 57.5 | 73.5 | 2.22 | 200 | | | |
| F. | " 4 | Clear | 30 | 55.0 | 85.0 | 0.08 | 3 | 2 000 | 16.5 | 8.2 |
| R. | " 5 | 20 | 16 | 67.5 | 83.5 | 2.23 | 500 | | | |
| F. | " 5 | Clear | 31 | 60.0 | 91.0 | 0.12 | 12 | 2 550 | 19.0 | 8.7 |
| R. | " 6 | 15 | 11 | 52.5 | 63.5 | 1.18 | 200 | | | |
| F. | " 6 | Clear | 32 | 57.5 | 89.5 | 0.09 | 5 | 2 900 | 18.0 | 9.0 |
| R. | " 7 | 15 | 15 | 62.5 | 77.5 | 1.60 | 800 | | | |
| F. | " 7 | Clear | 31 | 60.0 | 91.0 | 0.06 | 7 | 2 200 | 18.0 | 9.0 |
| R. | " 8 | 12 | 14 | 72.5 | 86.5 | 1.80 | 800 | | | |
| F. | " 8 | Clear | 31 | 62.5 | 93.5 | 0.06 | 9 | 2 450 | 19.0 | 8.9 |
| R. | " 9 | 15 | 13 | 67.5 | 80.5 | 2.04 | 900 | | | |
| F. | " 9 | Clear | 30 | 62.5 | 92.5 | 0.09 | 12 | 2 300 | 18.0 | 9.9 |
| R. | " 10 | 50 | 13 | 70.0 | 83.0 | 2.52 | 1 200 | | | |
| F. | " 10 | Clear | 32 | 67.5 | 99.5 | 0.07 | 22 | 2 450 | 17.0 | 12.0 |
| R. | " 11 | 150 | 15 | 50.0 | 65.0 | 4.6 | 3 100 | | | |
| F. | " 11 | Clear | 31 | 57.5 | 88.5 | 0.1 | 16 | 1 900 | 17.0 | 12.8 |
| R. | " 12 | 140 | 16 | 60.0 | 76.0 | 3.00 | 1 500 | | | |
| F. | " 12 | Clear | 34 | 55.0 | 89.0 | 0.11 | 14 | 2 600 | 17.0 | 14.4 |
| R. | " 13 | 100 | 9 | 75.0 | 84.0 | 6.50 | 1 700 | | | |
| F. | " 13 | Clear | 14 | 70.0 | 84.0 | 0.12 | 14 | 2 400 | 17.5 | 15.4 |
| R. | " 14 | 100 | 9 | 75.0 | 84.0 | 5.06 | 1 900 | | | |
| F. | " 14 | Clear | 27 | 67.5 | 94.5 | 0.06 | 5 | 2 100 | 15.0 | 12.9 |
| R. | " 15 | 70 | 9 | 55.0 | 64.0 | 4.60 | 500 | | | |
| F. | " 15 | Clear | 24 | 65.0 | 89.0 | 0.11 | 16 | 2 200 | 18.5 | 10.6 |
| R. | " 16 | 50 | 12 | 55.0 | 67.0 | 3.20 | 600 | | | |
| F. | " 16 | Clear | 17 | 50.0 | 67.0 | 0.1 | 24 | 2 700 | 8.0 | 9.4 |
| R. | " 17 | 50 | 12 | 55.0 | 67.0 | 2.52 | 1 100 | | | |
| F. | " 17 | Clear | 20 | 60.0 | 80.0 | 0.11 | 11 | 2 500 | 8.5 | 9.2 |
| R. | " 18 | 40 | 16 | 62.5 | 78.5 | 3.00 | 1 200 | | | |
| F. | " 18 | Clear | 22 | 60.0 | 82.0 | 0.15 | 23 | 2 000 | 8.0 | 8.8 |
| R. | " 19 | 30 | 16 | 57.5 | 73.5 | 2.92 | 100 | | | |
| F. | " 19 | Clear | 25 | 60.0 | 85.0 | 0.1 | 18 | 2 300 | 8.5 | 9.2 |
| R. | " 20 | 15 | 14 | 65.0 | 79.0 | 1.06 | 1 300 | | | |
| F. | " 20 | Clear | 22 | 70.0 | 92.0 | 0.06 | 10 | 2 100 | 9.5 | 9.2 |

R. Indicates river water. F. Indicates filter water.

REPORT OF TESTS OF THE WATER FROM THE FILTRATION PLANT AND THE OHIO RIVER, FOR JANUARY, 1914.—(*Continued.*)

| | Date. | Turbidity. | Alkalinity. | Incrustants. | Total hardness. | Iron. | Bacteria per cubic centimeter. | Rate of pumping. | Vacuum. | Stage of river, in feet above low water. |
|---|---|---|---|---|---|---|---|---|---|---|
| R. | Jan. 21 | 10 | 11 | 62.5 | 73.5 | 2.32 | 300 | | | |
| F. | " 21 | Clear | 22 | 67.5 | 89.5 | 0.08 | 5 | 2 250 | 8.0 | 10.0 |
| R. | " 22 | 25 | 14 | 57.5 | 71.5 | 3.52 | 1 500 | | | |
| F. | " 22 | Clear | 22 | 60.0 | 82.0 | 0.11 | 5 | 2 250 | 8.0 | 11.5 |
| R. | " 23 | 20 | 19 | 60.0 | 79.0 | 3.20 | 1 300 | | | |
| F. | " 23 | Clear | 26 | 67.5 | 93.5 | 0.1 | 1 | 2 250 | 6.5 | 13.0 |
| R. | " 24 | 100 | 11 | 72.5 | 83.5 | 7.04 | 1 900 | | | |
| F. | " 24 | Clear | 26 | 70.0 | 96.0 | 0.1 | 6 | 2 500 | 4.0 | 16.0 |
| R. | " 25 | 100 | 9 | 55.0 | 64.0 | 6.16 | 900 | | | |
| F. | " 25 | Clear | 22 | 57.5 | 79.5 | 0.19 | 4 | 1 900 | 5.0 | 15.8 |
| R. | " 26 | 90 | 10 | 52.5 | 62.5 | 4.00 | 400 | | | |
| F. | " 26 | Clear | 24 | 47.5 | 71.5 | 0.15 | 6 | 2 150 | 7.0 | 16.1 |
| R. | " 27 | 60 | 16 | 60.0 | 76.0 | 2.80 | 600 | | | |
| F. | " 27 | Clear | 27 | 55.0 | 82.0 | 0.11 | 4 | 2 100 | 9.0 | 16.0 |
| R. | " 28 | 60 | 13 | 60.0 | 73.0 | 3.62 | 500 | | | |
| F. | " 28 | Clear | 25 | 65.0 | 90.0 | 0.07 | 7 | 2 300 | 8.0 | 16.0 |
| R. | " 29 | 60 | 11 | 60.0 | 71.0 | 3.50 | 300 | | | |
| F. | " 29 | Clear | 25 | 55.0 | 80.0 | 0.07 | 4 | 2 650 | 9.0 | 15.5 |
| R. | " 30 | 60 | 12 | 52.5 | 64.5 | 4.00 | 300 | | | |
| F. | " 30 | Clear | 24 | 57.5 | 81.5 | 0.05 | 6 | 2 500 | 7.5 | 15.9 |
| R. | " 31 | 80 | 11 | 57.5 | 68.5 | 4.96 | 1 200 | | | |
| F. | " 31 | Clear | 24 | 60.0 | 84.0 | 0.14 | 6 | 2 200 | 4.0 | 18.1 |

R. Indicates river water. F. Indicates filter water.

# APPENDIX II

## Reports of Bacteriological Tests of Water of the Parkersburg Water-Works for the Year 1915.

By Lonzo O. Rose, M. D.

| Date of collection, 1915. | Source. | Bacteria per cubic centimeter. | *Bacillus coli* on following sowings: | | |
|---|---|---|---|---|---|
| | | | 1/10 c. c. | 1 c. c. | 10 c. c. |
| Jan. 23.... | River.... | 604 | — | — | + |
| " 23.... | Filter.... | 3 | — | — | + |
| " 24.... | River.... | 800 | — | + | + |
| " 24.... | Filter.... | 3 | — | — | — |
| " 25.... | River.... | 500 | + | + | + |
| " 25.... | Filter.... | 5 | — | — | + |
| " 26.... | River.... | 200 | — | + | + |
| " 26.... | Filter.... | 2 | — | — | — |
| " 27.... | River.... | 300 | + | + | + |
| " 27.... | Filter.... | 4 | — | — | — |
| " 29.... | River.... | 200 | — | + | + |
| " 29.... | Filter.... | 2 | — | — | — |
| Feb. 1.... | River.... | 1 200 | + | + | + |
| " 1.... | Filter.... | 12 | — | — | — |
| " 27.... | River.... | 200 | — | — | + |
| " 27.... | Filter.... | 12 | — | — | — |
| " 27.... | Tap.... | 18 | — | — | — |
| May 4.... | Pump Station.... | Sterile | — | — | — |
| " 4.... | Parkersburg Machine Co.... | 3 | — | — | — |
| Aug. 19.... | No. 1800 20th St.... | 2 500 dead end | — | — | — |
| Sept. 9.... | No. 1800 20th St.... | 11 " " | + | + | + |
| Oct. 10.... | No. 1303 St. Mary's Ave.... | 10 | — | — | — |
| " 10.... | 23d St. and Dickel Ave.... | 44 | — | — | — |
| " 10.... | No. 569 5th St.... | 7 | — | — | — |
| " 28.... | No. 1711 Park St.... | 18 | — | — | — |
| Nov. 1.... | No. 712 Grafton St.... | 86 | — | — | — |
| Dec. 20.... | No. 409 12th St.... | 2 | — | — | — |
| " 20.... | No. 932 Juliana St.... | 4 | — | — | — |

+ = present.
— = absent.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## DISCUSSION ON FLOODS AND FLOOD PREVENTION*

BY C. McD. TOWNSEND, M. AM. SOC. C. E.†

C. McD. TOWNSEND,‡ M. AM. SOC. C. E. (by letter).‖—In submitting a closure to the discussion of the Report of the Special Committee on Floods and Flood Prevention,¶ the writer has been impressed with the force of Bishop Warburton's remark that "Orthodoxy is my doxy". There is a tendency by various members of the Society to criticize the Committee for not giving special emphasis to the particular "ism" they advocate, without giving due consideration to the difficulties of formulating any report on which the Committee could agree. Mr. Townsend.

The subjects which were before the Committee have been discussed for many years, and most of its members had positive convictions which, if each had insisted on expressing, would have led to as many reports as there were members. A series of such essays would not have had much weight with legislators or the general public. Mr. Leighton lays particular stress on the necessity of engineers—and particularly of this great national society of civil engineers—influencing legislation. The only way such results can be obtained is by united action. It became necessary, therefore, for the Committee to seek some common ground on which its members could agree. It was the realization of the necessity of adjusting divergent views, if results of

* Discussion of the Progress Report of the Special Committee on Floods and Flood Prevention for 1915, continued from September, 1916, *Proceedings*.

† Closing discussion.

‡ St. Louis, Mo.

‖ Received by the Secretary, December 13th, 1916.

¶ Published in *Proceedings*, Am. Soc. C. E., for December, 1915, p. 2771 of Papers and Discussions.

Mr. Townsend.

value were to be obtained, not the shirking of duty, which led the Committee to make the safe and conservative report which Mr. Leighton criticizes.

Others criticize the report because it does not discuss the subject in sufficient detail. This phase of the subject was considered by the Committee, and it was found that the territory is so vast, the problems arising in various sections of the country so different, that either general principles, alone, could be enunciated, or a treatise of interminable length, based on incomplete data, would have resulted. A brief report, therefore, was agreed upon, leaving to the members the privilege of elaborating, in the discussion, such details as they considered advisable.

The science of river hydraulics is not in such a nebulous condition as Mr. Eakin appears to think, nor does the river engineer have to be guided solely by the classic work of Humphreys and Abbot. Although, possibly, American engineers have not investigated sufficiently the laws governing the flow of rivers, those of Europe have given the subject profound study. The researches of Fargue on the Loire, Girardon on the Rhône, Liliavski on the Dnieper, Hagen, Schlichting, and Jasmund on the rivers of Germany, and others, have revolutionized the science in recent years. Moreover, the Committee was not appointed to write a treatise on river hydraulics, and could assume that the rudimentary laws governing the flow of water in rivers were known to members of the American Society of Civil Engineers. It is evident from the discussion, however, that many do not realize sufficiently the differences between the laws governing the flow of water in pipes, sewers, and other channels having an immovable bed, and those which govern the flow in a stream having a bed which fills or scours with changes in velocity. Van Ornum's "Treatise on the Regulation of Rivers" and the second edition of Thomas and Watt's "Improvement of Rivers" throw considerable light on the subject, and are more suitable sources from which to seek information than a report on Flood Prevention.

In the minority report, a similar assumption is made. Although the majority of the Committee states that the physical data are lacking on many streams to enable them to solve the problem of flood control properly, they are not prepared to admit that "extensive studies are needed regarding the physical and physiographic laws affecting stream loads, rate of transportation and deposition and the variation of these factors with different conditions" before they can solve the problem if the physical data are supplied.

It is so self-evident that, at such times as the ground is frozen and trees are without foliage, these conditions prevent the utilization of reforestation or ground storage, that there is no necessity of making such a statement in a report. The point the majority of the Com-

mittee makes is that, if floods occur under such conditions, there is no need of discussing the effects of reforestation or ground storage. It would be just as pertinent to investigate the porosity of the soil under an asphalt pavement when designing a sewer to carry the water which flows over its surface. Mr. Townsend.

It is the writer's opinion that all investigations of the absorptive powers of different varieties of vegetation and the percolation through different kinds of soils are more valuable for studying problems of irrigation and drainage than those of flood control. Though it is considered perfectly feasible to measure these elements within a limited area and for a single river, the writer agrees with Gen. Chittenden that a determination for one situation, or season, or stream, would never apply reliably to any other. Moreover, to determine absorption of soils, it is necessary first to measure the run-off. When the discharge of a stream is determined, it is of no particular value to the problem of flood control whether the remainder of the rainfall has disappeared by evaporation or soil absorption.

The writer has recently investigated floods in Kansas, and they afford a pertinent illustration of this principle. During the summer the soil of Kansas provides a reservoir of enormous capacity, having a power of absorption which has been approximately measured by the Geological Survey. When the soil is prepared to receive it, a heavy rainfall will be accompanied by an exceptionally small run-off, but such conditions are of no value in determining flood prevention, as violent floods do not then occur. The floods in Kansas arise either from storms in February or March, when the ground is frozen and cannot absorb moisture, or in summer from a second storm which sweeps over the country within a few days after the reservoir capacity has been exhausted by a preceding rain. The run-off during ordinary conditions does not enter into the problem.

The majority report emphasizes the fact that reservoirs cannot be utilized simultaneously to reduce floods, to regulate the low-water discharge, and to increase the water-power that can be developed. The minority report seeks to minimize the effect of this statement.

There have been constructed at the head-waters of the Mississippi River a series of reservoirs primarily to increase its low-water flow. The practical manipulation of these reservoirs forcibly illustrates the necessity of the remarks in the majority report. Those who have meadows below the reservoirs demand that the flow be regulated so as not to interfere with their hay crop, those who raft logs want sufficient current to assist their business, and those who control water-power want a constant flow; in times of floods, those living above the dams are insistent that the gates be opened so as to reduce gauge heights on the upper river, and those below demand that the gates be closed to protect their property from overflow.

Mr. Townsend.

The statement that levees increase the reservoir capacity of the river channel appears to require further explanation. The construction of levees has increased flood heights on the Mississippi River between Cairo and New Orleans from 6 to 10 ft., and, as the levees are several miles apart, there is an enormous increase in the channel capacity during floods in this distance of 1 000 miles. As an illustration, the channel capacity above the overflow stage is given and compared with that of the Roosevelt Dam, but, as Gen. Chittenden states, the channel capacity of a river is by no means comparable in effect with that of a reservoir. The slope of the river is an important factor in the problem. The greater the slope the less being the influence of the channel capacity on flood heights; but, under favorable conditions, its influence is very large. Notwithstanding the inflow of tributaries, the maximum discharge of the Mississippi River at Vicksburg rarely equals that at Cairo, nor at New Orleans that below Red River Landing.

The straightening of a river has been a subject of discussion by European engineers for many years. Prior to 1890, German engineers generally adopted the idea that, in improving rivers for navigation, the channel should be given a uniform width, bends should be eliminated, or at least be given as gentle a curvature as practicable, and the river forced to a uniform slope, the natural gentle slope in pools in some cases being destroyed by submerged dikes extending out from the concave banks. The improvement of the Rhine, cited by Mr. Grunsky, and that of the Danube, cited by Gen. Chittenden, are examples of this type of improvement. It was also adopted by the Missouri River Commission for that portion of the Missouri River between Jefferson City and the Gasconade.

Difficulties, however, were experienced with this method of improvement. No matter how much care was exercised, the slopes through the improved reach could not be maintained as designed. At the upper portion, the water surface was gradually lowered, and a bar as invariably formed below it; an improvement in navigation was created through the reach, but the channel, both above and below, had less depth than existed formerly.

In a paper presented to the International Navigation Congress of 1894, on the "Improvement of the Rhône", M. Girardon called attention to these difficulties, citing as an example the portion of the Rhône called the Canal de Miribel.

He states:*

"The works were carried out according to this programme and finished in 1857; the resulting improvement was very evident and the navigation, in place of the continual uncertainty in which this passage left it, found in the canal of Miribel a channel easy to follow and

* Official Translation of the Congress, pp. 27 to 29.

of sufficient depth at any time of year. For several years this improvement appeared to be the only result of the works and nothing indicated that they could have any inconvenient consequences. Nevertheless the bottom of the canal was attacked; lower measurements began to be registered at low water than before the works were executed and these got lower and lower, while at the exit they marked higher and higher; then the foundation of the works, which had been made at low water-level, appeared in the upper portion of the canal and was even uncovered at the mean summer level of the water, while below the same water-level the tow-path was submerged and became useless throughout the greater part of the year. It became necessary to improve this situation which might at any time become very serious. The tow-path was first raised over half the length, that is the part submerged; then to arrest the movement it was decided to lower the dividing dike and to cut down the dam of Thil as low as possible, in order to allow the greatest possible volume of water to find its way through the false branches, not only in floods but also at the mean water-level. It was supposed that by this means the main branch would be relieved and that by decreasing the volume of water which crossed it its scouring action would cease. The result aimed at has been partially realized; the movement of the profile has become much slower, but has not entirely ceased; and although the volume of water which passes into the canal has become notably less, the incline [slope] which will set up any equilibrium between the resistance of the bottom and the effective force of the current under the new conditions of depth has not yet been attained. Mr. Townsend.

* * * * * * *

"Thus in spite of the improvement realized, in spite of the enormously extensive alterations that the river has undergone while passing between the constant banks, neither regularity in the cross-section has been obtained, nor constancy in the mean depth, nor uniformity in the incline [slope]; and the bed that has been opened remains formed as everywhere else of a series of hollows [pools] separated by ridges [bars], while the profile presents a series of reaches with a comparatively feeble incline [slope] separated by falls.

"Outside the canal the general incline [slope] of the combined streams at the entrance to Lyons, at kilometre 5, where the profiles join, has not sensibly varied; it was and still is 0.81 m. per kilometre, but its distribution has changed considerably. Throughout the canal, from kilometre 8 to kilometre 25, it was 0.883 m.; it is not more than 0.696 m. per kilometre on the other hand above the canal; from kilometre 25 to kilometre 34 it has been raised from 0.75 m. to 0.96 m., and below kilometre 8 to kilometre 5 it has been raised from 0.49 m. to 1.06 m. per kilometre."

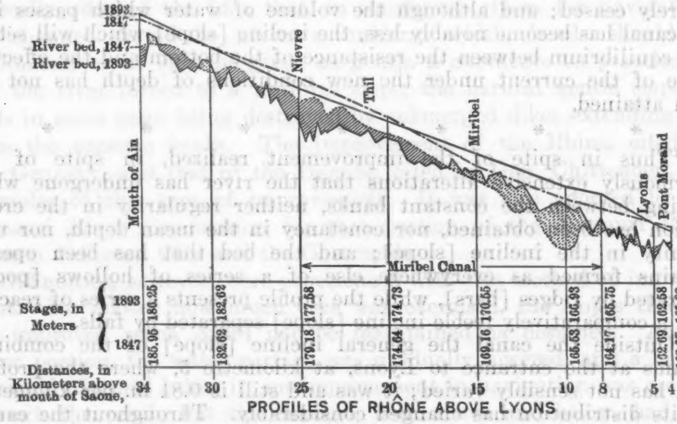
The profile, Fig. 6, derived from De Mas' "Rivières à Courant Libre", illustrates the condition.

Similar effects have been observed on western rivers of the United States. In the improvement of the Middle Mississippi below St. Louis the river was straightened for more than 20 miles. As a result, the slope was reduced at low water from above 0.6 to 0.25 ft. per mile,

Mr. Townsend.

and the low-water surface of the river at the Eads Bridge was lowered about 3 ft. An excellent channel exists through the improved reach, but the slope on the Chain of Rocks, above the city, has been increased, and dredging is required annually immediately below this reach in order to maintain the required channel depth. Similarly, the Kaskaskia cut-off, above Chester, which shortened the river about 6 miles, has a low-water slope through it of 0.23 ft. per mile, but dredging is required annually on the crossings above and below.

The improvement of the Missouri River reduced perceptibly the low-water plane at Jefferson City, and a cut-off at Camden Bend, which occurred in 1915, has created a bar 2 miles below it over which on November 3d, 1916, during an inspection trip made by the writer, a boat drawing 3½ ft. had to be sparred, the gauge at Kansas City reading 7.8 ft. At the bend above Omaha Mission, about 764 miles above the mouth, during the June flood of 1916, another cut-off occurred, and on October 20th the snagboat *McPherson,* drawing 3 ft., had to be sparred over shoals, both above and below this cut-off, the Omaha gauge reading 6.4 ft.




PROFILES OF RHÔNE ABOVE LYONS

FIG. 6.

On the Arkansas River, a very forcible illustration of the effect of cut-offs was also afforded during the flood of 1916. In prehistoric times the Arkansas River, by the caving of its banks, broke into the White River, about 10 miles above its mouth, and, until this season, the low-water discharge of the Arkansas emptied into the Mississippi through the mouths of the White. The cut-off between the two rivers gradually increased in length, and a sharp bend developed. During the flood of 1916 a cut-off occurred across this neck, resulting in such a shoaling in other portions of the old channel that at the present time

(November, 1916) the Arkansas River has resumed its old outlet into the Mississippi and now discharges its low-water flow in a separate channel from the White River. Mr. Townsend.

On the Red River numerous cut-offs have occurred, with results equally disastrous. Near Duke's Plantation, below Garland, the cut-off of 1915 created a bar below it over which at the next low water the depths of the navigable channel did not exceed 1 ft., and the efforts of the river to increase its length by caving after these cut-offs occurred have been destructive of the levee line which had been constructed along the river banks.

It is the raising of the river bed, not the increase of the discharge, which raises the water surface below cut-offs. The French engineers have called attention to this, and their views are now generally accepted in both Germany and Austria.

In his work on the improvement of the Rhine, Mr. Jasmund discusses the straightening of that river (cited by Mr. Grunsky) and explains that it is due to a previous undue reduction of slope, in former attempts to improve the river, that renders these cases exceptional, and conveys the impression that he disapproves of cut-offs on other portions of the Rhine.

The straightening of the Danube River in the vicinity of Vienna, quoted by Gen. Chittenden, did not improve the navigation of the river as much as was anticipated. About 3 500 000 cu. m. of material were scoured out of the cut, and deposited in the river below, raising its bed, and extensive work has since been required to adjust the low-water channel to these new conditions. It has even been necessary, in order to stabilize the bars, to create in the low-water channel new sinuosities to replace those cut out in the original improvement.

The view that a sedimentary stream seeks to regain its length after a cut-off did not originate on the Mississippi River, but is a corollary to views advanced by Guglielmini more than 200 years ago from observations he then made on the Po. Observations made recently by the Mississippi River Commission confirm this theory. Guglielmini's general principle, that there is a relation between discharge, slope, and the character of the soil, has also been verified by the changes which have taken place recently in the Atchafalaya and Illinois Rivers. In both these rivers a change in their discharge created by artificial means has been accompanied by a marked change in the radius of curvature of bends. A citation of an unreliable reconnaissance by Lewis and Clark is not considered sufficient proof to overcome the overwhelming evidence that can be cited to prove that rivers do strive to adjust their length to their discharge and slope.

Gen. Chittenden also takes exception to the distinction made by the Committee between streams created by glacial action and those carrying a large quantity of sediment. Although not agreeing with

Mr. Townsend.

Mr. Fuller, that there is a necessity for a new variety of engineer, called by him the geological engineer, the writer believes that all engineers who investigate river problems should have knowledge of geology. The methods of improvement which are successful with the gentle slope of rivers created by glacial action have frequently failed when applied to streams carrying large quantities of sediment. Thus, on the rivers connecting the Great Lakes a permanent channel of 21 ft. can be dredged, but on the Mississippi a channel dredged to 9 ft. fills annually. The permeable dikes, so successful on the Middle Mississippi, were failures when applied to the Upper Mississippi above Keokuk. On the Illinois River a cut-off could be made with impunity, but the writer believes it is impossible to improve the Missouri River permanently unless cut-offs are prevented.

In reference to outlets or spillways, the writer does not apprehend the danger to the channel of the main river from their construction that some do. He is a firm believer in the idea that rivers have a tendency to correct man's errors, and that, if outlets are constructed, a fill will occur in the outlets rather than in the river channel; but, if the outlet is given such dimensions that it begins to enlarge, there is imminent danger that the main channel will not only fill but be abandoned, unless the process is checked. A river exhibits an objection to serving two masters.

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# PAPERS AND DISCUSSIONS



## UNDERPINNING TRINITY VESTRY BUILDING FOR SUBWAY CONSTRUCTION

### Discussion.*

BY H. DE B. PARSONS, M. AM. SOC. C. E.†

H. DE B. PARSONS,‡ M. AM. SOC. C. E. (by letter).‖—The writer will endeavor to make clear some of the points raised in the discussion. Mr. Parsons.

Replying to Mr. Harte: The rebound of a concrete pile occurred as soon as the pressure was removed. In other words, the top of the pile rose while the hydraulic pressure was being relieved from the jack. When all the pressure was relieved, the rebound ceased.

Replying to Mr. Connelly: Concrete piles used in subway underpinning work, as described in the paper, are designed to carry safe loads, but are tested to loads in excess of the permanent safe loads. Thus, a building pier might support a load of 200 tons. If each pile were intended to carry 40 tons, there would have to be at least five piles beneath the pier. In such a case, each pile could be safely tested to a load in excess of 40 tons and be held in place under such a load. When the last pile was tested, it would temporarily relieve the excess loads from the others.

Although there is nothing in the paper which refers to a pressure of 77 tons on an empty pile made up of short steel sections, pressures as great as this are sometimes developed in sinking the empty steel casings.

Replying to Mr. Branne: Each jill-frame consisted of three 6 by 12-in. timbers, making the wooden part about 36 in. in width, sup-

* Discussion of the paper by H. de B. Parsons, M. Am. Soc. C. E., continued from November, 1916, *Proceedings*.

† Author's closure.

‡ New York City.

‖ Received by the Secretary, January 2d, 1917.

Mr. Parsons. ported on an iron frame composed of two 12-in., 40-lb. I-beams, about 18 in. centers, as shown in Fig. 10. Only one jill-frame was pushed forward at a time, and the center frame was kept slightly in advance of the side frames. The pressure used in moving a frame naturally varied owing to local resistances. The frames were moved by 3½-in., hydraulic rams, and the average pressure to start a frame was about 5 000 lb. per sq. in., but this fell, as the frame moved, to about 3 000 lb. per sq. in.

Referring in general to the remarks on the rebound of the concrete piles: It can be stated that the rebound was not attributable in any way to looseness of the joints of the steel casings. The function of the steel casings was to hold back the sand and to act as a form for the concrete. The loading was placed on the concrete and not on the casing.

The rebound may be due in part to the elastic recovery of the concrete, as mentioned by Mr. Buel, although his estimate for the modulus of elasticity is probably somewhat low. The rebound apparently was caused by the particles of sand under and around the foot of the pile readjusting themselves as the pressure on the pile was reduced. This "elastic recovery" of the sand and the "elastic recovery" of the concrete are probably both intimately connected with the rebound, although with present knowledge it is not possible to tell how much is due to the one and how much to the other.

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# PAPERS AND DISCUSSIONS



## A METHOD OF DETERMINING A REASONABLE SERVICE RATE FOR MUNICIPALLY OWNED PUBLIC UTILITIES

### Discussion.*


BY MESSRS. W. G. IRVING AND H. A. WHITNEY.


W. G. IRVING,† ESQ. (by letter).‡—The writer has attempted to give due consideration to the conclusions expressed in this paper, but has no doubt that more study would be profitable. He agrees with the author that a municipally owned utility should be viewed in the same light, and should have the same restrictions and privileges, as a privately owned public utility. Mr. Irving.

The construction or operation of a public utility, such as water-works, lighting works, etc., is not a municipal function. A city, as such, has no inherent right to engage in such forms of activity.

It is merely by special dispensation of the sovereign power that municipalities have been permitted to enlarge the scope of their powers to include the ownership and operation of utilities; and, in the operation of these utilities, with the exception of the fixing of rates, the municipality is impressed with the same obligations, as to service and as to responsibility for damages, as is a private owner of a public utility.

Therefore, the writer is of the opinion that the municipally owned public utility, in fairness to the privately owned public utility, should be under the same rate-fixing authority, and that the fixing of the municipal rates should take into consideration the same factors as are considered in fixing the rates of privately owned public utilities.

* Discussion of the paper by J. B. Lippincott, M. Am. Soc. C. E., continued from December, 1916, *Proceedings*.

† Riverside, Cal.

‡ Received by the Secretary, December 20th, 1916.

Mr. Irving. The writer, however, notes in this paper that the author has defined four classes of people who should contribute to the cost and maintenance of a municipally owned public utility. The writer is inclined to the opinion that this classification is arbitrary. No such classification is made with reference to privately owned public utilities. The city should either authorize a bond issue or make a direct appropriation from its funds, to provide the capital necessary to construct the utility. Then rates should be fixed by the constituted authority, to be charged the consumer for the service to be rendered, which should provide sufficient return to pay interest on the investment, cost of operation and maintenance, and to cover depreciation. There should not be, however, any amount allowed as a sinking fund to retire the bonds, if such are issued. The bonds should be retired by a direct tax on the taxable property of the city. As the bonds are retired, the interest payable thereon is decreased, and a greater proportion of profit of operating the utility should be paid into the general fund of the city which would permit a *pro tanta* reduction of taxes.

By this method, the owner of vacant property will be compelled to pay his portion of the cost of the system, and will also derive benefit therefrom, by reason of the enhanced value of his property and the ultimate reduction of his taxes by the profits earned by the utility.

The vacant lot owner, and all others who do not receive service from the utility, will also have to contribute to the maintenance of the system and to the payment of interest on bonds (if such are issued), during the adolescent period of the utility, as during that period the actual consumer cannot be expected to pay more than a fair rate, having in view the ultimate service to be rendered and the present capital expenditure.

By pursuing the foregoing method, a fair division of the general expenses of a publicly owned utility could be made, and, at the same time, it would prevent unfair competition with privately owned public utilities.

In the foregoing the writer has considered the question on general principles only, and has not had, and naturally could not have, in mind the many practical difficulties and scientific factors which would readily enter into the author's consideration of the question.

Mr. Whitney. H. A. WHITNEY,* M. AM. SOC. C. E. (by letter).†—Inasmuch as the underlying principles that go toward setting the service rate for one municipally owned public utility may be applied quite readily to all municipally owned utilities, the writer will confine his remarks to the utility of greatest importance, *viz.*, "the water department".

* San Francisco, Cal.

† Received by the Secretary, December 28th, 1916.

In order to estimate the annual charges against a water system thus owned, the following items should be known: Mr. Whitney.

*Schedule I.*

(1)—The reproduction cost of the system, less the depreciation;
(2)—The annual depreciation of the system;
(3)—The annual cost of operation and maintenance;
(4)—The cost of business management (generally included in (3));
(5)—The amount of interest to be paid annually;
(6)—The amount to be set aside for new construction.

These annual charges will be divided into the following classes:

*Schedule II.*

Item 1.—Charges to the municipality; bond issues for construction;
" 2.—Charges to local improvement districts for local construction;
" 3.—Charges to the municipality to be paid from general funds;
" 4.—Charges to consumers for service installations and extensions;
" 5.—Charges to consumers for domestic rate;
" 6.—Charges to consumers for manufacturing purposes.

The principal advantages gained by a municipality in owning its utilities are:

*Schedule III.*

(1)—Lower interest rates than those generally allowed privately owned utilities;
(2)—Lower water rates to consumers, due to vacant property helping to pay for the bond issues, and other charges paid for by the municipality;
(3)—Privilege of having ample fire protection, and thereby getting:
(4)—Lower insurance rates;
(5)—Power of extending the system where it can do the greatest good to the greatest number of people.

The foregoing are generally cited by advocates of a municipally owned utility as benefits derived by the municipality which owns its own system. Although, in a good many cases, a low water rate can be obtained by the consumer at the expense of the entire city (it bonding itself to pay the sinking fund and interest on the cost of the whole system, and the construction being permanent), yet, as in the case of the City of San Diego, the city was quite willing to do this, as was shown in 1912, when, by a vote of 6 to 1, the city purchased the

Mr. Whitney. impounding system, and again, in 1914, when, by a vote of more than 5 to 1, it decided to purchase Morena Dam. Furthermore, the heavy taxpayers were all back of the project.

Santa Rosa, Cal., went even further than this, a few years ago, and, by a large majority, voted bonds to purchase and operate a water system—the consumers to have free water.

The total annual revenue and charges should be such that they will equal the last five items in Schedule I. A list of these charges is set forth in Schedule II, and will be taken in their regular order.

*Item 1.—Charges to the Municipality; Bond Issues for Construction.*—As a main trunk sewer, a main drainage canal in a municipality, or a trunk highway, is a benefit to a community as a whole, so should dams, main pumping plants, and transmission lines be so considered; and the costs should be paid by the municipality, by either a tax levy or a bond issue.

*Item 2.—Charges to Local Improvement Districts for all Local Construction Which is not a Benefit to the Community as a Whole.*—Under this item all distributing mains, service mains, local reservoirs, and stand-pipes would be assessed against the district benefited.

The following sections from Chapter 98, Sections 3, 5, and 15, of the Local Improvement Laws, State of Washington, for 1911, will illustrate quite aptly the privileges derived by being able to apply these laws. The following are extracts from certain portions of these laws:

Section 3 allows the city to provide for sewer, drain, and water systems, within or without the city limits.

Section 5 allows the city to provide for protection from fire by an auxiliary water system. Besides allowing the construction of the ordinary distribution system, the following paragraph allows the construction of drains of any size, either within or without the city limits.

"Section 15.—Any city shall have power to provide for the construction of trunk sewers, and trunk water mains, and for the payment of all or any part of the cost and expense thereof by the levying and collecting of assessments upon property specifically benefited thereby. In any such case the district created to bear such assessment shall be outlined in conformity with topographical conditions, and in case of trunk sewers, shall include as near as may be all the territory which can be sewered or drained through such sewer and sub-sewers connected thereto, and in case of water mains, shall include as near as may be all the territory in the zone or district to which water may be distributed from such trunk water main through lateral service and distributing mains and services. In distributing such assessments, there shall be levied against the property lying between the termini of the improvement, back to the middle of the blocks along the marginal lines of the street or area improved, such amounts as would represent the reasonable cost of a local sewer and its appurte-

nances, or a water main and its appurtenances suited to the requirements of such territory in the mode prescribed in Section 13 hereof, and the remainder of the cost and expense of such improvement shall be distributed and assessed against all the property within the bounds of said entire district in accordance with the special benefits conferred thereon and in proportion to area." Mr. Whitney.

Under these local improvement laws, the City of Seattle and the City of Tacoma put in their distributing systems. Seattle uses only cast-iron pipe, and 8 in. as the minimum diameter. Tacoma uses 6 in. as a minimum diameter. The costs of all hydrants are assessed equally on the district.

*Item 3.—Charges to the Municipality to be Paid from the General Funds.*—Under this head will be:

(*a*) Charges to the fire department for the maintenance and operation of hydrants, and, where the cost of the system has been paid for out of water receipts, a charge should be made for overbuilding it from what would be necessary as a domestic supply to that of a "fire protection system". The only way to arrive at this proportion is to make a thorough study of the pressures needed, and the pipe sizes adequate for a domestic supply, and then place a value on the system thus estimated. Then calculate the cost of the water system necessary for an adequate fire protection, both as to pressure and supply. The difference in cost between a system designed for adequate fire protection and one which is necessary only for a domestic supply, will be the amount on which the municipality should pay for interest, depreciation, and maintenance. Roughly, for a city of 50 000 people, the cost of fire protection service would constitute about 50% of the cost of the entire system.

(*b*) The school board should pay the cost of all water used in schools at the same rate as regular domestic consumers.

(*c*) The park board should pay, without any discount, the cost of all water used in parks.

(*d*) The board of public works should pay the cost of all water used for streets, sewers, or in any municipal department under its jurisdiction.

*Item 4.—Charges to Consumers for Service Installations and Extensions.*—In most cities a charge is made to a new consumer for putting in the house service and the meter. This is generally a fixed charge.

*Item 5.—Charges to Consumers for Domestic Rate.*—The domestic rate is generally ascertained, as a lump sum, by deciding that all other items shall be met out of the water revenue, and then adjusting the

Mr. Whitney. charges of the domestic and manufacturer's rates so that it will equal this amount.

*Item 6.—Charges to Consumers for Manufacturing Purposes.*—Items 5 and 6 are generally worked in conjunction with one another. After all else has been fixed, the revenue necessary to be derived from the domestic and manufacturer's rates can be determined. The only rational and satisfactory method of serving water to the public is by meter. Under a strict meter rate, it is known at all times how much water is being used and for what purposes. The Peoples Water Company, of California, serves the Cities of Oakland, Berkeley, Alameda, Richmond, Piedmont, and several small towns with water. The system is 100% metered. The writer had occasion to take from the books of this utility the quantities of water that were being used for various purposes in Berkeley during 1915, that city having about 12 000 service connections. It was found that, out of 100% of the live consumers, the distribution of the percentages were as shown by Schedule IV:

*Schedule IV.—Percentages of Consumers Using Various Quantities of Water.*

| | |
|---|---|
| Consumers using 400 cu. ft. or less. . . . . | 41.7% |
| " " 500 " " " . . . . . | 54.2% |
| " " 800 " " " . . . . . | 77.4% |
| " " 1 000 " " " . . . . . | 84.7% |
| " " 5 000 " " " . . . . . | 99.1% |
| " " 50 000 " " " . . . . . | 99.9% |
| " " more than 50 000 cu. ft. . . | 0.1% |

The distribution of the various quantities is shown in Schedule V:

*Schedule V.—Percentages of Quantities of Water Used.*

| | |
|---|---|
| Quantity used, up to and including 400 cu. ft. | 15.0% |
| " " " " " 500 " " | 22.7% |
| " " " " " 800 " " | 42.0% |
| " " " " " 1 000 " " | 50.4% |
| " " " " " 5 000 " " | 81.6% |
| " " " " " 50 000 " " | 93.0% |
| " " in quantities of more than 50 000 " " | 7.0% |

In setting rates in a municipality, it is often the desire of the community to have the water consumers pay a low rate, and do so, to a great extent, at the expense of the property owners. It often happens that there is, besides the municipal water department, a privately owned utility which serves the community with water, and, unless some provision is made to take care of the rates of the utility,

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in order that they may parallel those of the municipality, a discrimination will be worked on the privately owned system. In order to illustrate how to establish a low rate which will work fairly to both parties, the writer will assume a system designed for fire protection, the distributing system to be laid by local improvement districts, the collecting and transmission systems to be paid for out of the taxes or bonds, and the maintenance and operation, and emergency construction to be paid for out of the water receipts. The following data are assumed for illustration:

| | |
|---|---|
| 1.—Population | 50 000 |
| 2.—Number of consumers | 10 000 |
| 3.—Annual consumption, in cubic feet | 200 000 000 |
| 4.—Percentage of water used per month less than 1 000 cu. ft. | 23 |
| 5.—Percentage of consumers using less than 1 000 cu. ft. per month | 54 |
| 6.—Percentage of water used per month, between 1 000 and 2 000 cu. ft. | 27 |
| 7.—Percentage of water used per month, between 2 000 and 10 000 cu. ft. | 31 |
| 8.—Percentage of water used per month, between 10 000 and 40 000 cu. ft. | 9 |
| 9.—Percentage of water used per month, more than 40 000 cu. ft. | 10 |
| 10.—Estimated present cost of system | $3 000 000 |
| 11.—Interest on bonds at 4½% per annum | $ 135 000 |
| 12.—Depreciation on system at 2% | $ 60 000 |
| 13.—Cost of operation and maintenance | $ 110 000 |

It is contemplated that the interest shall be paid out of the general tax on the system, and that the depreciation (either by sinking fund or otherwise) shall be paid from the water receipts, as will also the cost of operation and maintenance. This makes a total:

| | |
|---|---|
| 14.—For depreciation, and maintenance and operation | $ 170 000 |

*Schedule VI.—Assumed Rates.*

| | |
|---|---|
| 15.—For 1 000 cu. ft. per month or less | $ 1.00 |
| 16.—Cost per 100 cu. ft. from 1 000 to 2 000 cu. ft. | 0.10 |
| 17.—Cost per 100 cu. ft. from 2 000 to 10 000 cu. ft. | 0.7½ |
| 18.—Cost per 100 cu. ft. from 10 000 to 40 000 cu. ft. | 0.5 |
| 19.—Cost per 100 cu. ft. for more than 40 000 cu. ft. | 0.3 |

From Items 1 to 9 and 15 to 19, a table of rates (Table 5) has been compiled.

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TABLE 5.—RATES OF RETURN, AS INDICATED BY ITEMS 1 TO 9 AND 15 TO 19.

| | |
|---|---|
| 54% of consumers at $1.00 per month (using 23% of the water per annum) | $64 800 |
| 27% of 200 000 000 cu. ft. at 10 cents per 100 cu. ft. per annum. | 64 000 |
| 31% " " " " " 7½ " " " " " " | 46 500 |
| 9% " " " " " 5 " " " " " " | 9 000 |
| 10% " " " " " 3 " " " " " " | 6 000 |
| Total revenue | $180 300 |

Table 5 shows that the revenue amounts to an annual rate of $18 per consumer. If it is decided to construct $100 000 worth of improvements to the distributing system and $200 000 worth to the collecting and transmission system, the distributing system to be improved by a local improvement district, bonds at 6%, and the collecting and transmission system to be improved with bonds against the municipality bearing interest at 5%, we have:

| | |
|---|---|
| $100 000 at 6% | $6 000 |
| 200 000 " 5% | 10 000 |

Making a total amount borne by the city, as follows:

| | |
|---|---|
| $3 000 000 at 4½% (as shown by Item 11) | $135 000 |
| 200 000 at 5% | 10 000 |
| Total municipal bond interest | $145 000 |
| Local improvement bonds | 6 000 |
| Maintenance, operation, and depreciation | 170 000 |
| Grand total for support of system | $321 000 |

Inasmuch as the expense of the bonds is borne by the municipality, this leaves only the maintenance and operation to be taken care of through the water revenues. The estimated revenue, as shown by Table 5, is $180 300, and the maintenance and operation being $170 000, there is left a surplus of $10 300 for emergency construction.

Some method should be adopted whereby the privately owned utility can charge a rate which will be approximately the same as that of the municipality, and at the same time pay a reasonable revenue to its owners. If it is the desire of the municipality that the water consumers shall have low rates at the expense of the taxpayers, it apparently should not matter (if the utility is serving as good water at as ample a pressure and in sufficient quantities as the municipality) whether the difference is paid by taxes on bonds against the municipal system or by taxes on the general funds to subsidize the plant of the utility.

An annual budget equal to the interest on the estimated cost of the privately owned utilities plant could be assessed against the general taxes and allowed to this utility as rental for the use of its system and the privilege of using the fire hydrants; the utility should be allowed to pay—as mentioned in the case of the municipality—the maintenance, operation, and depreciation expenses, from the revenue derived from the sale of water, this revenue to be regulated by the city council or other authorized body. Thus competition would be placed on an equal basis. Mr. Whitney.

The foregoing is a suggestion as to some of the ways in which revenue might be derived. Numerous modifications of these methods may be made, such as charging rentals for hydrants, street sprinkling, sewer flushing, etc. Inasmuch as all of these are paid for from monies derived from the general taxes, it amounts to the same in the end. Then, too, the revenues may be increased to the extent of having the department pay the expense of construction.

By the methods outlined herein a municipality can set a rate which will place as small or as large a burden on the water consumer as may be desired. The water must have a fixed cost, and what the consumer does not pay the taxpayer must.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852


# PAPERS AND DISCUSSIONS



## THE VALUATION OF LAND

### Discussion.*


By Messrs. W. I. King, Hugh A. Kelly, Edward S. Rankin, and T. Kennard Thomson.


W. I. King,† Esq. (by letter).‡—There is a dangerous and time-honored economic fallacy hidden in the sentence "The true value of land is the ground rent capitalized". After two thousand years of search for the "true value", economists are in agreement that no such thing exists. Furthermore, "ground rent capitalized", unless the phrase is broadly interpreted, gives no kind of value of any significance. Mr. King.

Every prospective buyer or seller of land has in mind a maximum price which he will pay or a minimum price which he will take. These subjective prices are finally based on the beliefs as to the probable future income which may be expected from the land. In other words, each person, using his own rate of interest, discounts these anticipated future rentals and finds their entire present worth. With their subjective prices in mind, bidders enter the market, and, by the well-known principles of supply and demand, fix the market price for the land.

At some periods, nearly every one becomes unduly optimistic as to the future outlook; at other times pessimism reigns. This change in the point of view of the majority causes land values to fluctuate widely on the market. If these values are plotted, it will be noticed, however, that there is a rather definite trend in a certain direction. This trend records values arising from the normal consensus of opinion, and may be said to represent normal land values; but value is always based on anticipation, and, until the veil is rent from the future, no

* This discussion (of the paper by L. P. Jerrard, Jun. Am. Soc. C. E., published in November, 1916, *Proceedings*, and presented at the meeting of December 20th, 1916), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof. of Political Economy, Univ. of Wisconsin, Madison, Wis.

‡ Received by the Secretary, December 14th, 1916.

Mr. King. man will ever be able to state correctly that any figure represents the true value of the land.

In a progressive community, land values usually stand well above the capitalized present net rentals, because people do not expect that rentals will remain constant, but rather that they will rise. To correct for this error by varying the rate of interest used in capitalizing the current rent is, at the best, a very crude method of adjustment. The writer has described, in "The Valuation of Urban Realty for Purposes of Taxation",* ways of solving the problem which appear to be more logical.

The writer notes, also, an apparent misconception of the word "economic" as used on page 1375.† The demand of wealthy persons for pleasant residence sites is certainly as strictly economic as the demand of a railway for a site for a new depot or roundhouse.

Mr. Jerrard's discussion of such topics as the movement of the business section, "plottage", "cost of acquiring land", and "values based on special utilization", is both able and interesting, and appears to be better developed than any writings on the subject which the writer has previously read. The author is to be congratulated on the scientific and scholarly nature of the paper as a whole.

Mr. Kelly. HUGH A. KELLY,‡ ASSOC. M. AM. SOC. C. E.—This paper is a valuable addition to our records. The speaker, being in constant touch with the Taxing Department of Jersey City, has become familiar with the question of "land valuation", and when he read this paper he realized that it was practically flawless. In order to be sure, however, he handed it to the two men whom he thought best qualified to judge: Mr. J. Stewart Walker, formerly Chief Engineer of the Jersey City Map Department, and Mr. Raymond C. Buckley, the present Chief.

Jersey City issued last year a manual for the guidance of the working force in the Bureau of Tax Assessment of the Department of Revenue and Finance. This "Assessors' Manual", the first issued by any city in the United States, was written by these gentlemen, under the direction of Commissioner George F. Brensinger, and after reading Mr. Jerrard's paper, they also agreed that he has left little to discuss. Nevertheless, the following few points may be well taken.

If one examines cases of tax appeal one will find that the real estate man must have his builder, his architect, to support him. In the more important cases, such as railroad cases, it is the engineer who knows real estate who is called on to testify. In a town the speaker has in mind he knows of but two real estate men who have carefully compiled their sales and put them in proper form for immediate use in appeal cases.

* *Bulletin 689* of the University of Wisconsin. May be obtained from the Librarian of the University.

† *Proceedings*, Am. Soc. C. E., for November, 1916.

‡ Jersey City, N. J.

This only emphasizes Mr. Jerrard's contention that the engineer should check the real estate man, and verifies the statement that the average real estate dealer's appraisal is but a look, a thought, an interview, and an opinion. Mr. Kelly.

The author states:*

"The ground rent is the net income which remains after paying the taxes on the land and the taxes, depreciation, interest, and maintenance on all improvements."

To this list of deductions there should be added the item "a sinking-fund charge sufficient, when put at interest, to cover the cost of the building in a given number of years".

How many real estate men include this item for their clients when presenting cases?

The author states that "the true value of land is the ground rent capitalized".

First: True value is spoken of by Judge Charles C. Black, in his book on "Taxation" in which he reviews many judicial decisions, thus:

"True value, like fraud, is incapable of definition, for the reason that no definition can have universal application to all the many classes of property."

He concludes with the statement:

"So the 'true value' of property, under the constitution for taxation, comes back in all cases to what such property will exchange for in the open market, in money. Sometimes cost is a fair criterion; earning capacity may be; frequently both should have weight; while, again, neither will determine its 'true value'; but all these elements of value should be considered."

Second: If one wishes to calculate the value of land from an income basis, one must select adequately improved property. Then if one adds to the present worth of all future anticipated net rentals of a new building, the present worth of the land value at the date when the building will need to be replaced, and deduct from the sum the cost of the building, it should closely approximate the present true value of the site.†

However, as new buildings exactly adapted to their location are rarely found, and the difficulties with this method are considerable, the sales method is to be preferred to the commercial method of finding the value.

Rarely is there an appraisal of property in which a true capitalization method is used.

In the report of the Morris Canal Investigation Committee, appointed by the joint Legislature of New Jersey, Louis Focht, Assoc.

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* *Proceedings*, Am. Soc. C. E., for November, 1916, p. 1373.

† King on "Valuation of Urban Property."

Mr. Kelly. M. Am. Soc. C. E., Engineer of State Board of Taxation, uses such a method.

He projects a possible development, assuming 2 050 ft. of bulkhead, two piers 150 ft. wide, about 2 060 lin. ft. in all. On each pier will be placed one-story steel sheds 1 175 by 113 ft. and 1 250 by 140 ft., respectively; also a six-story warehouse covering an area of 83 000 sq. ft., together with the necessary tracks, filling back of bulkhead, dredging of slips, etc.

"The commercial value has been based on the capitalization of the net surplus after deducting the probable annual fixed charges from the probable annual income.

"The annual fixed charges are made up of the interest on the investment, an annuity sufficient to create a sinking fund for the purpose of extinguishing the debt against the perishable portion of the property at the termination of its probable life; taxes on the property, annual charge for dredging slips to maintain a sufficient depth of water; insurance on the portion of the property likely to be destroyed by fire; current annual repairs; and superintendence.

"These elements will now be considered in their order:

"First: In arriving at the value of the probable annual income, rentals of dock property in the harbor have been considered. They are lower on the New Jersey than on the New York side, and range about as follows:

| | | | | |
|---|---|---|---|---|
| New Jersey side of harbor...... | $0.32 | to | $0.40 | per sq. ft. |
| New York side of harbor...... | 0.20 | to | 1.54 | per sq. ft. |
| Average................ | | | 0.87 | per sq. ft. |

"Brooklyn rental, private piers, range from $0.48 to $0.57 per sq. ft. Warehousing rates range from about $0.25 to $0.30 per sq. ft. of floor surface.

"In estimating the probable income, a rate of $0.50 per sq. ft. has been assumed for dock property, and $0.25 per sq. ft. of floor surface for the warehouse. On this basis, the following result is obtained:

PROBABLE ANNUAL INCOME.

| | |
|---|---|
| Docking, 309 000 sq. ft. at $0.50 per sq. ft............ | $154 500 |
| Warehouse floor area, 498 000 sq. ft. at $0.25 per sq. ft. | 124 500 |
| "Total annual income................ | $279 000 |

"Other minor sources of income might be added, but, for the sake of conservatism, they have not been considered.

"The rate of $0.50 per sq. ft. is higher than the average New Jersey rentals, the development being of a higher character than other leased properties on the New Jersey side.

"An estimate of the probable cost of the development has been made, in order to arrive at the fixed charges.

"The estimated cost of the entire development for the 'Small Basin' is $2 265 000.

"In determining the annuity to be set aside for the sinking fund, an assumed life had to be given to each depreciable portion of the plant.

These values will vary with the character of the structure. Considering an interest rate of $3\frac{1}{2}\%$ on the sinking fund, the annuities in Table 3 are obtained. Mr. Kelly.

TABLE 3.

| | Life. | Per thousand. | On: | |
|---|---|---|---|---|
| Bulkhead | 50 years | $7.63 | $153 750 | $1 173 |
| Piers | 40 " | 11.83 | 618 000 | 7 311 |
| Sheds on piers | 40 " | 11.83 | 400 000 | 4 732 |
| Warehouse | 50 " | 7.63 | 755 300 | 5 763 |
| Paving | 15 " | 51.82 | 9 750 | 505 |
| Gantry cranes | 15 " | 51.82 | 30 000 | 1 555 |
| Tracks | 15 " | 51.82 | 37 811 | 1 959 |
| Totals | | | $2 004 611 | $22 998 |

"In the computation of the annuities in Table 3, it will be noticed that there is no item for grading or other permanent work, which, when once completed, would last indefinitely.

"On the foregoing basis, therefore, the statement in Table 4 will represent the entire annual charges.

TABLE 4.—ANNUAL CHARGES.

| | |
|---|---|
| Interest on $2 265 000 at 5% | $113 250 |
| Annuity for sinking fund on $2 004 611. at $3\frac{1}{8}\%$ (average) | 22 998 |
| Taxes on $1 698 750 at $2.00 per hundred | 33 975 |
| Annual charge for dredging slips to maintain constant depth of water, 20 000 cu. yd. at $0.20 | 4 000 |
| Insurance on $1 800 000 of dock property, at 0.5 of 1% | 9 000 |
| Current annual repairs | 15 000 |
| Superintendence | 5 000 |
| Total fixed annual charges | $203 223 |

"With the fixed annual charges shown in Table 4, the net surplus will be:

Total annual income.......................$279 000
Annual fixed charges.......................203 223

Balance, or annual net surplus........ $75 777

"Therefore, if the present worth of this net surplus, namely, $75 777, is computed on the basis of 5% interest for a term of 60 years, or the limit of the period when the canal will revert to the State, a value of $1 434 459 is obtained for the land."

The author (page 1374*) states as follows: "advantages of location govern the values entirely". The last word of this statement is open to question. The "Assessors Manual" quotes Mr. Alfred D. Bernard of the U. S. Fidelity and Guaranty Company, thus:

* *Proceedings*, Am. Soc. C. E., for November, 1916.

Mr. Kelly.

"We therefore announce four cardinal factors of value in real estate:

"1—Location, which includes access;
2—Utility, which includes capacity to produce;
3—Shape;
4—Size."

These are explained as follows:

"(1) Location.—In any city the most valuable corner lot would appear to be the lot in that city accessible to the most people.

"Now, if that proposition is sound, the most valuable corner lot is the center of population of any given city.

"In the City of Baltimore, the center of population is in the colored belt, at a point where land values are around fifty cents a square foot; so, in fixing the most valuable corner lot we must assume something else besides accessibility of location; we must assume the second factor, utility.

"(2) The highest utility which land may assume, generally speaking, is retail business, and the student of values will find that, with the exception of the banking district of New York City, the highest valued properties in all the cities may be found in the retail business districts.

"Now, with Factors 1 and 2 to resolve, we find the most valuable corner lot in any city is that lot which is so located as to be accessible to the greatest number of people who buy goods.

"After fixing the point, we must then consider the reason why one lot is more valuable than another, which calls upon us to invoke the other principles, of shape and size. The most desirably located corner may be too small to meet the business demand, and the business be carried to the next block, or, by reason of angles and irregularities, it might not be available for the business of the territory, and thus lose in contrast with other regular lots."

From the foregoing it is evident that the greatest number of people necessarily do not pass through the heart of the city, as Mr. Jerrard states, but more generally the greatest number of people pass near some railroad terminal or boat landing.

The retail section moves in the direction of the best residence section. Bernard sees it as follows:

"To-day the most valuable block in a given city is Main Street between X and Y Streets. Thomas Tenant's lease is about to expire. He has made big money in the twenty years he has engaged in the dry goods business in this block. His landlord wants to share his business with him and doubles his rent. Tom has made money and saved it, and feels independent enough to tell his landlord to go where ice is popular but scarce; but the landlord doesn't melt, and Tom has to pay double the rent or move. The district broker whom Tom employs to buy the site is met with 'Not for Sale', and tells Tom about four little shacks he can buy in the second block north of him and build his own building and laugh at the landlord. He

does it. He builds a fine warehouse, better than the one he was in, advertises to the trade that he was forced out of his old location, and thus wins sympathy and trade. He is followed by another unfortunate, and in three years the best block on Main Street is Tom Tenant's block." Mr. Kelly.

Mr. Jerrard states that the total land value of a city depends primarily on the population, but is influenced by wealth.

Bernard says:

"While the proposition may be new and startling, the writer's study of land values in cities warrants him in asserting that land values bear a relation to the number of and buying power of the inhabitants of a given community, and that in the average normal city the value of the retail business property is one cent per foot front per person and the best residence property 1/10 to 1/12 of a cent per foot per person."

In answer to the disadvantage of using gross rentals, Bernard uses such rentals, and gives, as the advantage, the fact that it eliminates the item of poor management, which invariably brings down the net rental.

If the value of the improvement is 9 times the value of the site, the gross rental should be 19% of the total investment,

| | | | | | | |
|---|---|---|---|---|---|---|
| 8 | times | the | value | of the | site. | .18% |
| 7 | " | " | " | " | " | .17% |
| 6 | " | " | " | " | " | .16% |
| 5 | " | " | " | " | " | .15% |
| 4 | " | " | " | " | " | .14% |
| 3 | " | " | " | " | " | .13% |
| 2 | " | " | " | " | " | .12% |

Improvement and site equal (for office building only) 10 to 12 per cent.

To Mr. W. A. Somers is attributed the statement that the corner influence in all property generally extends 100 ft. each way from the corner. The literature of the Manufacturers Appraisal Company contained this statement, but that company acknowledged this to be an error. The speaker thinks it is an exceptional case where corner influence extends more than 25 ft. beyond the corner.

The value of lots of various depths, expressed as percentages of unit value, or "rules for long and short lots", as it is termed, requires the attention of a mathematician.

The speaker believes that Mr. Davies is the only man who gives a formula with his rule, and states that the curve is a parabola.

Interpolation, to determine intermediate values, is not satisfactory where many properties are handled. In the Jersey City Tax Office, to facilitate matters, the engineers in the Map Department have an

Mr. Kelly. equation for the Hoffman rule, which is in the Tax Manual published by the Department of Revenue and Finance of that city.

It is stated that Mr. Davies examined 10 000 sales. Mr. Bernard, Mr. Somers, and all the others examined many sales in order to derive their curves of values.

There is a question in the speaker's mind whether a man can be so familiar with every one of 10 000 sales as to be able to squeeze out all the influences that would pull them away from normal. Probably the law of averages would counteract errors in so great a number of sales.

The problem of depth and value presents an example resulting in an equation the curve of which, when plotted, is that of a conic section. The Hoffman rule works out as a parabola.

Knowing these results, why is it not as good to assume a condition, as Hoffman did, and work out the equation; or why not take the least number of sales or points that will determine a conic curve, and if the curve suits our judgment, adopt it.

If all the values in a curve do not respond to some formula, the speaker claims that the curve is theoretically incorrect, as no two persons are treated according to the same law.

Would it surprise the author to know that Hoffman's opinion, in 1870, agrees with Bernard's careful investigation, in 1913? If Mr. Jerrard's conversion of the Bernard 150-ft. to 100-ft. rule is correct, such is the case. Unfortunately, Hoffman's complete figures are not given in Table 2. In Table 5 is given a comparison of Hoffman's and Bernard's figures. In no case is there a difference greater than 1 per cent.

TABLE 5.

| Depth of lot, in feet. | HOFFMAN. | BERNARD. |
|---|---|---|
| | Percentage. | Percentage. |
| 5 | .... | 10 |
| 10 | 16 | 17 |
| 15 | 23.5 | 24 |
| 20 | 30 | 31 |
| 25 | 37.5 | 37 |
| 30 | 44 | 43 |
| 35 | 50 | 50 |
| 40 | 56 | 56 |
| 45 | 61.5 | 61 |
| 50 | 67 | 66 |
| 55 | 71.5 | 72 |
| 60 | 76 | 76 |
| 65 | 80 | 81 |
| 70 | 84 | 84 |
| 75 | 87.5 | 87 |
| 80 | 91 | 91 |
| 85 | 93.5 | 93 |
| 90 | 96 | 95 |
| 95 | 98 | 98 |
| 100 | 100 | 100 |

In the author's comparison of the rules, it will be noticed that, up to 100 ft., the Newark business is the same as the Somers. The Newark residence, up to 100 ft., is practically the same as the Hoffman. There is a close resemblance up to 100 ft. of Milwaukee business and Somers, and for more than 100 ft. they agree. For more than 100 ft. Newark business and residence agree. Mr. Kelly.

Such peculiarities would lead one to suppose that some of the rules represent no independent investigations.

In Table 6 the speaker examines the Hoffman rule by subtracting the value of every 10 ft.: ten lower from ten higher: thus 10 ft. = 25%; 20 ft. = 41%; 41 — 25 = 16, etc. This gives a series of percentages or first differences.

TABLE 6.—EXAMINATION OF THE HOFFMAN RULE.

| Feet. | Percentage. | First differences. | Subtract. | Second differences. |
|---|---|---|---|---|
| 10 | 16 | 15 | 10 | 5 |
| 20 | 31 | 13 | 10 | 3 |
| 30 | 44 | 12 | 10 | 2 |
| 40 | 56 | 11 | 10 | 1 |
| 50 | 67 | 9 | 10 | — 1 |
| 60 | 76 | 8 | 10 | — 2 |
| 70 | 84 | 7 | 10 | — 3 |
| 80 | 91 | 5 | 10 | — 5 |
| 90 | 96 | 4 | 10 | — 6 |
| 100 | 100 | | | |

If 10 is subtracted from each of these we get a series of second differences in which it will be noted that 4 is lacking.

The original Hoffman rule does not plot as a smooth curve, but if 4 is supplied in this last series of numbers the following formula may be derived.

Assuming that 50 ft. gives $66\frac{2}{3}\%$, the formula is:

$$y = \frac{2\,x}{3} - \frac{x}{150}.$$

If 50 ft. is assumed to give 67%, the formula is:

$$y = 1.68x - 0.0068x,$$

which gives a true parabolic curve.

These formulas were derived by Mr. Walker. This peculiar condition leads to the belief that some real estate men do not understand

Mr. Kelly. these rules thoroughly; for, in important cases, where large tracts of land are involved, we find them analyzed by real estate men thus:

Zone 1, so many feet deep, parallel with the street, is worth *a* dollars. Zone 2, directly behind it, is worth *b* dollars, etc.

But Zone 1 reflects its value to Zone 2, etc., and immediately the straight-line valuation begins to bend, and, the smaller the zones are divided, the greater becomes the reflected increments, until the straight-line valuation becomes some kind of a curve.

The speaker believes that these rules for short and long lots are the final result of this line of reasoning, and the foregoing examination of the Hoffman rule proves it.

The appraisal of lands in different localities and sections requires individual attention and study, and for these no rules can be laid down which will fit all cases. Mr. Jerrard's paper gives a firm foundation on which to build.

Mr. Rankin. EDWARD S. RANKIN,* M. AM. SOC. C. E.—This discussion refers to the following statement in the paper:

"The total land value of a city depends primarily on the population, but is influenced to so great an extent by the city's wealth and other factors that comparisons between cities of the same population, as to either the total land value or the value of highest priced land, offer considerable variation. It is believed that the range of values in Fig. 2 will cover the conditions in all except a very small percentage of cities."

In an attempt to formulate the relation between the population and land values in cities, the speaker some years ago addressed letters to real estate dealers in the 109 cities of the United States having a population of 50 000 or more, asking the value per front foot of the highest priced land in their respective cities, based on a standard lot 25 by 100 ft., exclusive of corners. Replies were received from 47 cities, the smallest being Canton, Ohio, with 50 200, and the largest, St. Louis, with 687 000 population (census of 1910).

The results obtained, though of a negative character, may prove of some interest in connection with this paper. They have been plotted on a diagram, Fig. 4, similar to Fig. 2, and seem to show the hopelessness of attempting to base valuations on population, except in the most general way. It would appear, however, that the curves in Fig. 4 are entirely too low.

Reduced to dollars per front foot per 1 000 population, the variations are perhaps even more apparent. Some of the main features of the investigation, reduced to this basis, are shown in Tables 7 and 8.

Believing that the rate of growth of the city would have a decided bearing on the valuations—on the principle that values would discount

* Newark, N. J.

the future—the speaker then made an estimate of the probable population of each of the cities in 1920, and calculated the value per front foot per 1 000 of this estimated population. The results are given in Table 8, and show slightly less variation, but are still too far apart to be of any practical value. Mr. Rankin.

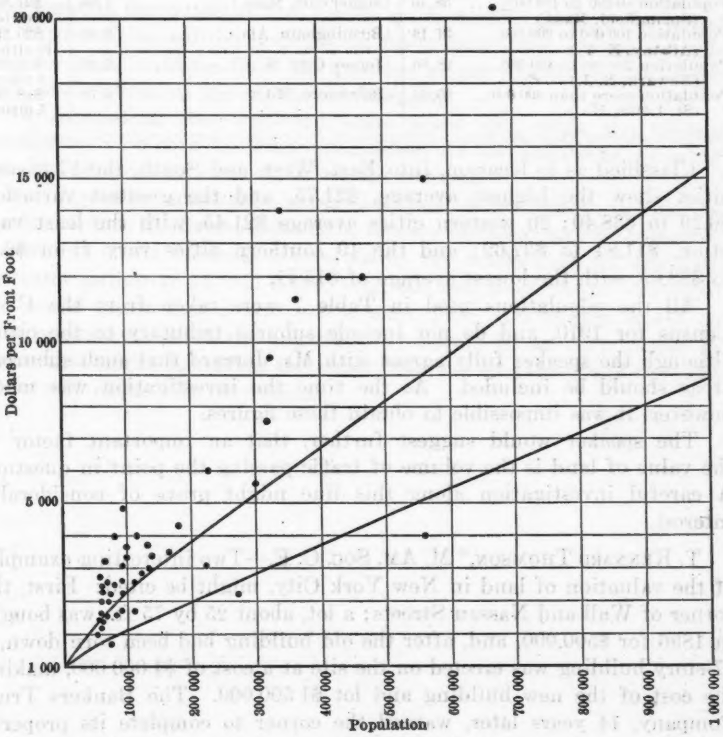



FIG. 4.

TABLE 7.

| Maximum. | | Minimum. | | Average. |
|---|---|---|---|---|
| All cities | $54.00 | (Baltimore, Md.) | $7.16 | $27.86 |
| (Springfield, Mass.) | | | | 47 cities. |
| Population 50 000 to 100 000 | 54.00 | (Somerville, Mass.) | 9.71 | $30.85 |
| (Springfield, Mass.) | | | | 24 cities. |
| Population 100 000 to 200 000 | 35.52 | (Nashville, Tenn.) | 15.80 | $26.64 |
| (Grand Rapids, Mich.) | | | | 10 cities. |
| Population 200 000 to 400 000 | 40.28 | (Jersey City, N. J.) | 8.06 | $23.12 |
| (Newark, N. J.) | | | | 8 cities. |
| Population more than 400 000 | 29.48 | (Baltimore, Md.) | 7.16 | $23.59 |
| (St. Louis, Mo.) | | | | 5 cities. |

Mr. Rankin.

TABLE 8.

| Maximum. | | Minimum. | | Average. |
|---|---|---|---|---|
| All cities (Springfield, Mass.) | $38.40 | (Jersey City, N. J.) | $6.29 | $20.33 47 cities. |
| Population 50 000 to 100 000 (Springfield, Mass.) | 38.40 | (Somerville, Mass.) | 7.65 | $21.33 24 cities. |
| Population 100 000 to 200 000 (Albany, N. Y.) | 31.13 | (Birmingham, Ala.) | 9.00 | $20.29 10 cities. |
| Population 200 000 to 400 000 (Newark, N. J.) | 28.86 | (Jersey City, N. J.) | 6.29 | $18.77 8 cities. |
| Population more than 400 000 (St. Louis, Mo.) | 25.00 | (Baltimore, Md.) | 6.78 | $18.10 5 cities. |

Classified as to location, into East, West, and South, the 17 eastern cities show the highest average, $21.75, and the greatest variation, $6.29 to $38.40; 20 western cities average $21.45, with the least variation, $11.84 to $37.62; and the 10 southern cities vary from $6.78 to $23.69, with the lowest average of $15.71.

All the populations used in Table 7 were taken from the U. S. Census for 1910, and do not include suburbs tributary to the cities, although the speaker fully agrees with Mr. Jerrard that such suburban areas should be included. At the time the investigation was made, however, it was impossible to obtain these figures.

The speaker would suggest further, that an important factor in the value of land is the volume of traffic passing the point in question. A careful investigation along this line might prove of considerable interest.

Mr. Thomson.

T. Kennard Thomson,* M. Am. Soc. C. E.—Two interesting examples of the valuation of land in New York City, might be cited: First, the corner of Wall and Nassau Streets; a lot, about 25 by 75 ft., was bought in 1896 for $500,000, and, after the old building had been torn down, a 17-story building was erected on the site at a cost of $1 000 000, making the cost of the new building and lot $1 500 000. The Bankers Trust Company, 14 years later, wanted the corner to complete its property in order to have about 90 ft. front on each street, so the 17-story building and lot was bought for $1 500 000 and then $50 000 was paid to a wrecker to remove the building quickly.

The second case is that of a young man who had an assured income of $25 000 a year; he painted a little, wrote a little, and tried to make himself generally useful, and at the same time was able to live comfortably on his income. An uncle bequeathed him a $1 000 000 piece of property on West Street. Just then, unfortunately, the tenant moved out, and a new one could not be found, and as the property was subject to a mortgage which required nearly all the young man's $25 000 income to pay, he was absolutely impoverished by the bequest

* New York City.

of a \$1 000 000 piece of real estate, which he could neither rent nor sell. Mr. Thomson.

One of the great stumbling blocks in modern real estate is the old 25-ft. front lot, which was all right in the days of one- or two-story cottages and stores, but is absolutely out of place in a modern city, and is responsible for much of our "crazy quilt work". It is to be hoped that future cities will be laid out on up-to-date lines, giving a whole block for each building, then the question of the relative values of the inside and outside of the block will disappear.

An interesting case occurred some years ago, when the caisson foundations of a new building encroached on the adjoining property at depths of from 30 to 90 ft. below the surface.

Mr. Jerrard's admirable paper has one defect which should be remedied before the final publication, thereby obliterating this comment thereon. It is the impression received by the reader that the engineer is a better authority on real estate than the real estate man. No engineer would grant that a real estate man, pure and simple, could do better engineering than the engineer. It should be an easy matter to remove this slight flaw from the paper, and the speaker trusts that the author will make this correction.

# MEMOIRS OF DECEASED MEMBERS.

**NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.**

## ELMER LAWRENCE CORTHELL, President, Am. Soc. C. E.*

DIED MAY 16TH, 1916.

Elmer Lawrence Corthell was born in South Abington (now Whitman), Mass., in 1840, the son of James Lawrence and Mary Gurney Corthell. He was educated at the South Abington High School and Phillips Exeter Academy. He entered Brown University in 1859, studying until the outbreak of the Civil War in 1861. At the first call for volunteers, he enlisted in the First Rhode Island Artillery, Battery D, and served in stations from private to the rank of Commander of field artillery, until his return with rank of Captain at the end of the war. As a soldier, he saw service principally in Virginia and North Carolina.

At the close of the war Mr. Corthell returned and resumed his studies at Brown University from which he was graduated in 1867 with the degree of Master of Arts, and Phi Beta Kappa honors.

In 1894 he received from his Alma Mater the honorary degree of Doctor of Science. From boyhood it had been Mr. Corthell's aim and ambition to study for the ministry in the Baptist Church, but on completing his university course, on account of his health, and the advice of his physician to select a more active profession, he chose Civil Engineering and entered the employ of the late Samuel Barrett Cushing, M. Am. Soc. C. E., a prominent engineer in Providence, R. I., where he practiced and did field work in railroad, mill, dam, bridge, city, and other construction.

In 1868 Mr. Corthell was appointed Assistant Engineer on the construction of the Hannibal and Naples Railroad in Illinois, now a part of the Wabash System. In 1869, he was Engineer on the location and construction of the Hannibal and Missouri Railroad. In 1870-71, he was Chief Assistant Engineer, under the late Col. E. D. Mason, M. Am. Soc. C. E., in the construction of the railroad bridge over the Mississippi River at Hannibal, Mo. From 1871 to 1875, he was Chief Engineer of the Sny Island Levee, 51 miles in length, on the east bank of the Mississippi River. While engaged in that work he was, in 1873-75, Chief Engineer on the construction of the bridge over the Mississippi River, at Louisiana, Mo., for the Chicago and Alton Rail-

* Memoir prepared by the following committee: John F. Wallace and J. A. Ockerson, Past-Presidents, Am. Soc. C. E., and W. J. Karner, Assoc. Am. Soc. C. E.

road. The draw-span of this bridge was 444 ft. long—at that time, the longest in the world.

In July, 1875, Mr. Corthell went to the mouth of the Mississippi River, and from that time until 1880 was Assistant to the late Capt. James B. Eads, M. Am. Soc. C. E., in the construction of the jetties at the South Pass mouth of the river. On account of ill-health, he came north early in 1880, and while convalescing, wrote his "History of the Jetties at the Mouth of the Mississippi River".*

From 1881 to 1884, Mr. Corthell served as Engineer of the New York, West Shore and Buffalo Railroad Company, in charge of the construction of that road and the extension of the New York, Ontario and Western Railroad to connect with the West Shore. For about two years, during the construction of the West Shore Railroad, he was assisting Capt. Eads and Col. James Andrews in their plans for the construction of a ship railway across the Isthmus of Tehuantepec, Mexico, and, after he resigned his position with the West Shore Railroad Company, from 1884-87, he assisted Capt. Eads in promoting the Tehuantepec Ship Railway, taking charge, on the Isthmus, of the survey of the route for the railway, and exploiting it by visiting various cities in the United States, showing perfect working models of the railroad and a ship being placed in its cradle on the rails.

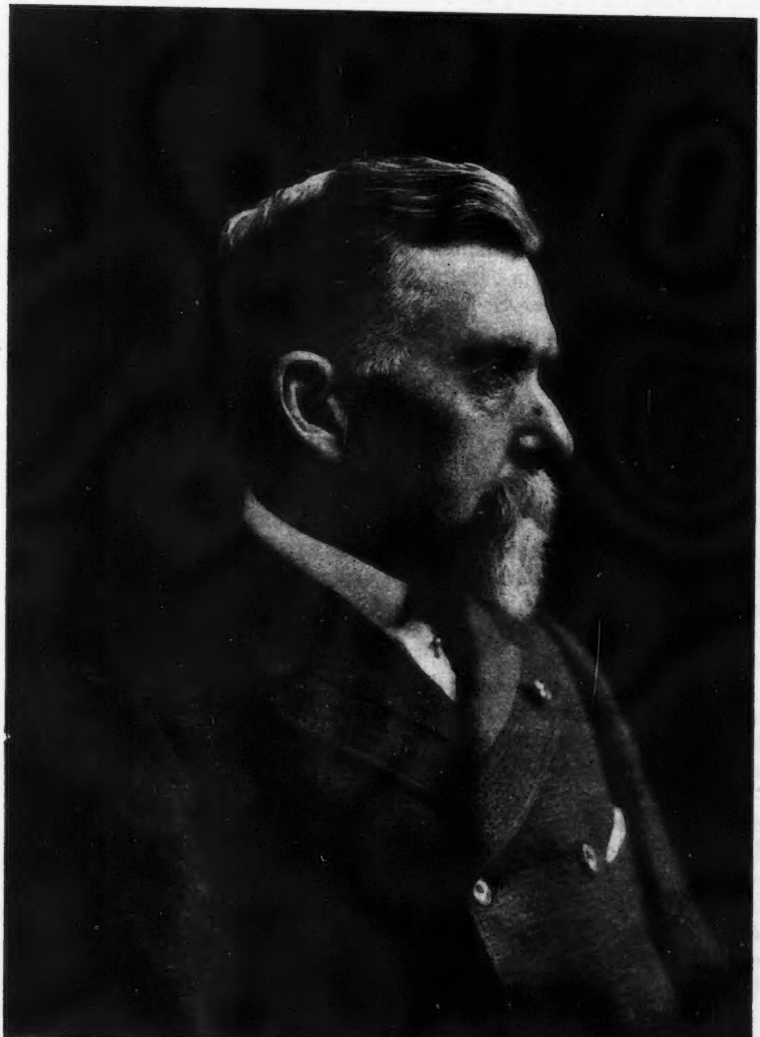
From March, 1887, until 1889, Mr. Corthell was associated in an engineering partnership, in New York City and Chicago, Ill., with the late George S. Morison, Past-President, Am. Soc. C. E., engaged in the construction of railroads, bridges, harbors, and water-works. During this partnership there were constructed, the Cairo Bridge over the Ohio River for the Illinois Central Railroad, then the longest steel bridge in the world, bridges over the Missouri River at Nebraska City and Sioux City, two bridges in Oregon, one at Jacksonville, Fla., and water-works at Bismarck, N. Dak.

From 1889 to 1890, he was Engineer of the St. Louis Merchants Bridge over the Mississippi River, having charge of the design and construction of the substructure and foundations.

During the same period, from 1889 to 1890, he was also Chief Engineer of the improvements at the mouth of the Brazos River, in Texas.

In 1889 and 1890 he was employed as a special Consulting Engineer in charge of terminal work in the City of Chicago for the Chicago, Madison and Northern Railroad, a subsidiary corporation of the Illinois Central Railroad,—the Chicago, Madison and Northern Railroad affording an entrance to Chicago from the West for the Illinois Central lines; and also in connection therewith for certain main and side-track facilities for the Atchison, Topeka and Santa Fe and the Chicago and Alton Railroads.

* Published in 1880 by John Wiley and Sons.



E. L. Corthell

In 1889, Mr. Corthell made examinations, plans, and report on the proposed improvement of the harbor at Tampico, Mexico, for the Mexican Central Railroad and, later, had charge of the construction of the jetties as Chief Engineer.

During 1889 he was also President and Chief Engineer of the Southern Bridge and Railway Company, incorporated that year to build a bridge over the Mississippi River, at New Orleans, and completed the plans and specifications for its construction.

In the following year (1890), he made a personal examination between the Great Lakes and Quebec, Canada, of the question of an enlarged waterway between Chicago, Duluth, and other ports of the Great Lakes and the Atlantic Seaboard, and wrote a paper on this subject for the Canadian Society of Civil Engineers* and the Western Society of Engineers at Chicago.

In 1891 Mr. Corthell visited Europe, with several important objects in view. As a Trustee of the University of Chicago, he examined six of the leading universities and technical schools of Europe in order to obtain information for the University in carrying out its purpose of establishing, in connection with it, a great School of Engineering and Architecture. As a member of a Committee of the Western Society of Engineers, engaged in solving the difficult railroad problem of Chicago, he examined in Europe thirty-five railroad terminals and complicated situations. He also examined twenty-six European harbors to secure special information for use in connection with his work at Tampico, Mexico, and elsewhere. He also examined nearly all the subways of the Old World, from Glasgow to Budapest.

In 1897, Dr. Corthell again visited Europe to examine a great variety of engineering works. Many of the results of his various examinations and investigations were published in the *Engineering Magazine*, in New York and London. The most extensive work done by him, however, during his two years in Europe, was on the subject of Maritime Commerce, its past, present, and future. In August, 1898, he presented the results of his work to the American Association for the Advancement of Science, which held its Fiftieth Anniversary, at Boston, Mass. The object of the paper was to show the development of commerce in the half century to come. On his return to the United States he was engaged as expert on several important works in the United States and Mexico.

Dr. Corthell sailed for Buenos Aires, Argentine Republic, in March, 1900, where, for more than two years, he was engaged in solving problems for commerce and reporting to the Minister of Public Works. Thirty-six different subjects were referred to him for investigation and report.

* *Transactions*, The Canadian Society of Civil Engineers, Vol V, p. 32 (1891).

During the winter of 1902 and the spring of 1903, he delivered thirty-six lectures in thirty cities of the United States and Mexico, on "Two Years in Argentine as Consulting Engineer of National Public Works". These were delivered before universities and commercial bodies, also engineering societies, etc., at the request of the Argentine Government.

At one time, Dr. Corthell made an examination, reports, and estimates for the Boston, Cape Cod, and New York Ship Canal.

The Governor of the State of New York, in 1894, appointed Dr. Corthell upon the Advisory Board of Consulting Engineers on the State Barge Canal, a position from which he resigned on account of his Brazilian work.

During 1904 and 1905 he was engaged on extensive commercial works in Brazil, at Para, in St. Catherina, and Rio Grande do Sul.

He represented the United States on the Permanent Commission of the International Congress of Navigation and was an active and influential member of that organization.

Mr. Corthell was married, in 1870, to Emily Theodate Davis, of Providence, R. I., who died in 1884, leaving two children, Mrs. E. S. Dewey, of Gloversville, N. Y., and Howard Lawrence Corthell, a Civil Engineer, of New York City. In 1900, he was married to Marie Kuchler, of Berne, Switzerland, who survives him. He also leaves one brother, Roland, of Boston, and one sister, Mrs. Annie C. Phipps, of Wollaston, Mass.

The various International Engineering Congresses always found Dr. Corthell an active participant, either in person or by important papers. At a meeting in Brussels, where all but six of the seventy-one papers were in other languages than English, he prepared for the Department of State, which had selected him as a delegate, a résumé of the entire proceedings, which formed a volume of 245 pages. The International Engineering Congress held during the Columbian Exposition was suggested by him, and its success was largely due to his work as Chairman of the Executive Committee having charge of that affair. When the recent Pan-American Scientific Congress, in Washington, D. C., was contemplated, the Department of State again called upon Dr. Corthell to aid in the organization of the meeting, and he rendered valuable assistance in making this affair a success.

After years of exceedingly active work, Dr. Corthell found his chief satisfaction in the fact that his works had been beneficial to commerce by sea, river, canal, and rail, and he could point with pride to the results as having aided in reducing the cost of transportation on land and water, to the benefit of mankind.

A prolific writer on engineering subjects, his printed papers fill many volumes. Another of his activities concerned the establishment

of a civilian reserve corps of engineers, and he was one of the leaders, if not the leader, in this project. After his election to the Presidency of the American Society of Civil Engineers, Dr. Corthell had taken an active part in the proposal to have the Society join the three other National Engineering Organizations in the United Engineering Societies Building in New York.

Dr. Corthell was a member of the following societies: American Institute of Consulting Engineers, Inc., of which he served as President in 1915-16; The Canadian Society of Civil Engineers; The Institution of Civil Engineers of Great Britain; The Society of Arts of Great Britain; Member d'Honneur, and Corresponding Member of the French Society of Civil Engineers; The American Association of Civil Engineers and Architects; American Railway Engineering Association; The Boston Society of Civil Engineers; The Western Society of Civil Engineers, Chicago, Ill., of which he was President in 1889; Honorary Member of the Geographical and Statistical Society of Mexico; American Geographical Society; The National Geographical Society, Washington, D. C.; Fellow, Royal Geographical Society, London, England; Fellow, American Association for the Advancement of Science; Honorary Member, Engineering Society of Portugal, The Institution of Engineers of the River Plate, The Centre de Navigacion Transatlantica, and Sociedad Cientifica of Argentine; Franklin Institute, Philadelphia, Pa.; American Highway Association; Pan-American Society; Founder, Pan-American Chamber of Commerce; Member, Chamber of Commerce of the United States; and Member, Board of Consulting Engineers, Barge Canals, New York State.

Dr. Corthell was also a member of the following military and patriotic associations: Grand Army of the Republic; Military Order of the Loyal Legion; Sons of the American Revolution; New England Society; and Society of the Army of the Potomac.

He was also a member of several academical and university societies and clubs, including the University Club, New York City, Phi Beta Kappa and Sigma Chi Societies, and of the Engineers Club of Rio de Janeiro.

After a half century of continuous work devoted to the service of his fellow-man and the advancement and up-building of the Engineering Profession, he has now passed away, leaving the Profession and the world better and enriched by his work, which has been a shining example of an enthusiastic, industrious, useful life.

Throughout it all, in adversity as well as in prosperity, he met his daily problems with enthusiasm, and ever had his hand extended to encourage and help all with whom he came in contact, particularly the younger members of the Profession. He was always ready to assist them, not only by advice and counsel, but by financial aid and the use of his influence, in obtaining positions and advancements.

His memory will always flourish in the hearts of those who have been favored by his acquaintance.

Dr. Corthell was elected a Member of the American Society of Civil Engineers, on September 2d, 1874. He served two terms as Vice-President, in 1889 and 1893-94, and was elected President on January 19th, 1916.

## DAVID WEST CUNNINGHAM, M. Am. Soc. C. E.*

DIED MAY 11TH, 1916.

David West Cunningham, the son of Andrew and Abigail Leonard (West) Cunningham, was born on December 24th, 1829, in Boston, Mass. He was educated at Chauncy Hall School and the Lawrence Scientific School of Harvard University.

In 1848, Mr. Cunningham received an appointment in the Engineering Corps of the Boston Water-works, then building the Cochituate Aqueduct. In 1849, he was on railroad work on the Manchester and Lawrence Railroad and the Sullivan Railroad of the Connecticut River, from which he was transferred to the Ogdensburg and Lake Champlain Railroad. He was stationed first at Moline and afterward at Lawrenceville on the Moira Sub-division, and rode on the first locomotive through to Ogdensburg, which was driven by Mr. Charles L. Schlatter, then Chief Engineer. He was then employed on railroad work in Canada. The eastern half of the Ogdensburg and Lake Champlain Railroad at that time ran through primitive forest, and while at this work, he contracted inflammatory rheumatism which affected his eyes, and he was compelled to give up engineering work for the time being. In 1851, he joined a friend who was a merchant at Cienfuegos, Cuba, and sailed from Boston for Spain early in June, arriving at Cadiz in 24 days.

After 6 months' travel in Europe, Mr. Cunningham entered the Lawrence Scientific School at Harvard, for a course in Civil Engineering, and was graduated as a Civil Engineer.

Mr. Cunningham spent six years in Chili, on railroad and government work, but left to be married. He never returned to that country, on account of his wife's preference for the United States. After leaving Chili, he operated a plantation in Texas.

After the Civil War Mr. Cunningham was engaged in the construction of the Charleston, Mass., Water-works, the Lowell Water-works, and the sewerage systems of Lowell, Mass., and Stillwater, Minn. He was Chief Assistant Engineer for the Boston Water-works. He built the Tarkio Valley (Missouri) Railroad, and later was Consulting Engineer for the Minneapolis Water-works.

* Memoir prepared by Frank H. Olmsted, M. Am. Soc. C. E.

Mr. Cunningham then worked a large wheat farm in North Dakota until 1894, at which time he removed to California.

He was married in 1859 to Mary B. S. Fuller, of Boston, who died in 1869. In 1873, he was married to Caroline Smith Thomas, who died at Los Angeles, Cal., in 1910. In 1913, he was married to Minnie A. Holderbaum, at Santa Ana, Cal.

Mr. Cunningham was a man of high scientific attainments, and naturally a builder. He was favorably known wherever his duties carried him. He was genial in disposition, loved his friends, and had few, if any, enemies. He could without doubt have reached a high position had he remained in the Profession and not digressed at one or two periods in his career to other kinds of work. He was a remarkably well read man, and spoke and wrote fine English. He was also an excellent Spanish scholar, and a valued member of the various scientific bodies to which he belonged.

Mr. Cunningham was a member of the Architects and Engineers' Association of Southern California. He was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

## SIDNEY WILLETT HOAG, Jr., M. Am. Soc. C. E.*

Died November 1st, 1916.

Sidney Willett Hoag, Jr., son of Sidney Willett and Ann Augusta Hoag, was born in New York City, on June 5th, 1857. He received his preliminary education in the public schools and was admitted to the College of the City of New York in 1872 as a student in the scientific course, from which he was graduated in June, 1877. From 1877 to 1879, he pursued the post-graduate course in Civil Engineering in the same college under the late Professor A. G. Compton, Assoc. Am. Soc. C. E., and was graduated with the certificate of Civil Engineer.

In 1879, after leaving college, Mr. Hoag went with the New York City and Northern Railroad, being first engaged on trestle construction. Subsequently, during construction, he had charge of the distribution of materials and supplies, and later was engaged in the construction of roadbed, turn-tables, and depots, from Croton Lake to Lake Mahopac.

In 1880 he entered the service of the City of New York, being engaged in the Department of Public Parks as an Assistant Engineer. His efforts in the Park Department were directed toward the public improvement of the annexed district. He had charge of various classes of engineering work under the jurisdiction of the Park Department, including surveys, laying out of streets, monumenting, preparation

* Memoir prepared by Charles W. Staniford, M. Am. Soc. C. E.

of tax maps, rule, damage, and benefit maps, and work of general construction involving regulating and grading, and, in addition, he directed the work of the various field parties.

In 1891, Mr. Hoag was appointed an Assistant Engineer in the Department of Docks and Ferries, and was assigned by George S. Greene, Jr., M. Am. Soc. C. E., then Engineer in Chief, as head of the drafting-room. In this position, Mr. Hoag directed the preparation of plans and specifications for various works of construction, undertaken by the Department along the water-front, such as piers, bulkheads, freight sheds, and freight and passenger terminals, and the approval of structural features of all plans submitted by private owners and lessees for all kinds of wharf structures.

In 1908, Mr. Hoag was appointed Deputy Chief Engineer of the Department under Charles W. Staniford, M. Am. Soc. C. E., Chief Engineer. During his incumbency as Deputy Chief Engineer, the Department of Docks carried out many important works of construction, involving piers, bulkhead walls, freight sheds, and ferry terminals. Some notable pieces of construction on which Mr. Hoag was engaged are the municipal ferry terminal at St. George, Staten Island, the municipal ferry buildings at the Battery, known as the Manhattan Terminal of the South Brooklyn Ferry, the Whitehall Terminal of the Staten Island Ferry, and also the viaduct approach to the St. George Ferry, at St. George. The most conspicuous and important undertaking by the Department of Docks during this period was the construction of the great transatlantic steamship terminal popularly termed the Chelsea Section. This is on the North River water-front, and covers the section from West 14th to West 23d Streets. It involved the construction of nine piers, with freight sheds thereon, and putting in heat, light, and power equipment for operating purposes. This section represents an expenditure of many millions of dollars, and the accommodation it affords for large transatlantic liners is not equalled anywhere in the world.

This large undertaking by the Department involved an enormous amount of detail work in connection with the preparation of plans and specifications, and the examination and approval of plans submitted by contractors during construction.

The successful completion of the Chelsea Section demonstrated Mr. Hoag's capacity for hard work, and his great attention to detail enabled the Department to carry out successfully many large undertakings involving extreme detail in steel construction and intricate problems in connection with heat, light, and power equipment. Mr. Hoag was a loyal assistant, conscientious in everything he did, and, above all, a man of sturdy integrity. His work was his hobby.

He never had any interest outside of his professional work, and was held in high esteem by the fellow members of his Profession.

In March, 1904, he became a member of the Society of Municipal Engineers of the City of New York, and at once took an active interest in its affairs. He was rewarded with honors for his labors, being elected Second Vice-President in 1910; First Vice-President in 1911, and President of the Society in 1912. He continued his interest in the Society, being on the Board of Directors as Past-President, at the time of his death.

In 1905, Mr. Hoag delivered, before the Municipal Engineers, a very interesting paper entitled "The Dock Department and the New York Docks." The paper dealt with the history of the Dock Department and its work, and contained considerable detailed data relative to water-front construction. It had a wide distribution, and was considered the most meritorious paper of the year. For this Mr. Hoag received the Society medal in recognition of his achievement.

In 1914, his health failed, and he was compelled by his physical condition to give up active work. He retired from the Department of Docks on a pension after 35 years in the service of the City.

After about a year's rest, he regained some of his strength and joined B. F. Cresson, Jr., M. Am. Soc. C. E., assisting the latter in the preparation of plans and specifications for the construction of an extensive water-front and railroad terminal development at Bayonne, N. J. He was associated with Mr. Cresson up to the time of his death.

System, the love of detail, and the habit of accumulating interesting data pertaining to his Profession, were some of Mr. Hoag's personal characteristics. He was a thorough, painstaking, and conscientious worker, and the bigger the job, the happier it made him. Although, in his younger days, he was fond of athletics and had joined the New York Athletic Club, engaging in running races, his whole time, during his later years, was given to the work of his Profession. He was a capable executive, had a frank, genial personality, a high code of honor, and won the admiration of those who knew him.

Mr. Hoag was a member of the New York Athletic Club, the Municipal Engineers of the City of New York, and the Royal Arcanum.

Mr. Hoag was elected a Member of the American Society of Civil Engineers on September 2d, 1885. He served as a member of the Nominating Committee in 1913-14.

## ROBERT MAITLAND ROY, M. Am. Soc. C. E.*

DIED JUNE 27TH, 1916.

Robert Maitland Roy, the son of Robert Maitland and Maria Roy, was born at Stirling, Ont., Canada, on November 27th, 1869. He was educated in the public and high schools of Belleville, Ont., and was

* Memoir prepared by the Secretary from information supplied by J. A. Spittle, Esq.

graduated from Ontario Commercial College. He also spent a year or more in the study of commercial and contract law, and took a special private course in mathematics.

In June, 1886, Mr. Roy entered the employ of the Grand Trunk Railway, serving, until October, 1888, in the Traffic and Engineering Departments.

From October, 1888, to February, 1892, he was engaged as Assistant Mechanical Engineer with the G. and J. Brown Manufacturing Company, of Belleville, Ont., inspecting Government and railway supplies, contractors' plant for public works, railway, mining, and other contracts, as well as structural work of various kinds. Mr. Roy was also engaged during this time on Government surveys of the Belleville Harbor and River protection works, on extensions to the Belleville Water-Works, and on new sewerage and pumping plants for the Government Institution for the Deaf and Dumb at Belleville.

From February, 1892, to January, 1898, Mr. Roy was employed with the Central Bridge and Engineering Company, of Peterborough, Ont., as Assistant and Contracting Engineer for railway and highway bridges and general structural work. He was also engaged during this time on surveys of the Trent Canal and other works.

In January, 1898, Mr. Roy entered the employ of the Hamilton Bridge Works Company, Limited, of Hamilton, Ont., serving as Highway Bridge Engineer from 1898 to 1908, as Manager, from 1908 to 1916, and as Director from 1913 to 1916, which latter positions he held at the time of his death, on June 27th, 1916.

Mr. Roy was a Member of the Canadian Society of Civil Engineers, the Hamilton Board of Trade, and the Canadian Manufacturers Association, and had served as an Alderman of Hamilton, Ont., from 1913 to 1916. He was also a member of the Hamilton, Twentieth Century, Canadian, and Hamilton Automobile Clubs, the I. O. O. F., the A. F. and A. M., and of the Church of England.

On April 19th, 1894, Mr. Roy was married to Mary Ann Wragg, daughter of Thomas B. Wragg, of Belleville, Ont., who, with four sons, survives him.

Mr. Roy was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1900, and a Member on March 5th, 1907.

## PAPERS IN THIS NUMBER

"CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL." A. C. Dennis. (To be presented Feb. 7th, 1917.)

"THE WATER SUPPLY OF PARKERSBURG, W. VA." William M. Hall. (To be presented Feb. 21st, 1917.)

## PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

# PAPERS AND DISCUSSIONS

FEBRUARY, 1917

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INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# UNUSUAL COFFER-DAM FOR 1 000-FOOT PIER, NEW YORK CITY

By Charles W. Staniford, M. Am. Soc. C. E.

To be Presented March 7th, 1917.

## Synopsis.

This paper is descriptive of the design and construction of a steel sheet-pile coffer-dam of unusual type; and, so far as known, it is the highest on record.

In outline the coffer-dam is a cellular steel sheet-pile structure, designed for a pressure head, of soft mud and water, of about 65 ft., having the cellular spaces filled with earth and clay to form the water seal, and held against the external pressure by an embankment of rip-rap or broken rock. All the steel sheet-piling forming the cellular spaces or "pockets" is in single pieces, many of which are more than 70 ft. long.

## Descriptive and Historical.

On the refusal of the Secretary of War to grant any further extension of the pierhead line in the Chelsea District, thereby preventing permanent additions to the present piers and thus making it impossible to dock the new large transatlantic liners—more than 900 ft. long—at that locality, the City of New York was obliged to secure another site

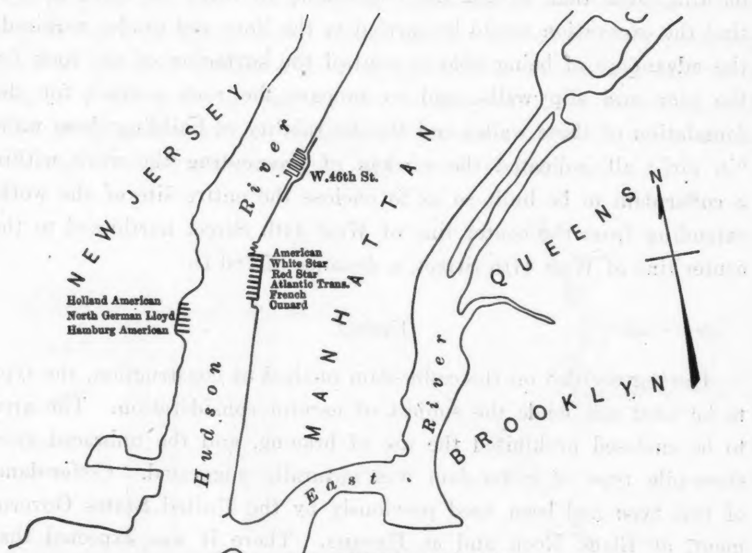
Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

which would permit the construction of piers long enough for such vessels; and on April 10th, 1913, the Commissioner of Docks, Mr. R. A. C. Smith, presented a plan providing for a pier 150 ft. wide, at the foot of West 46th Street, extending 1 000 ft. inshore from the established pierhead line, and a half-pier at the foot of West 44th Street, with a slip room of 360 ft. between them, and a temporary slip, on the north side of the West 46th Street pier, 237 ft. 6 in. wide. This plan was adopted by the Commissioners of the Sinking Fund on April 30th, 1913, and steps were immediately taken to acquire the necessary property.

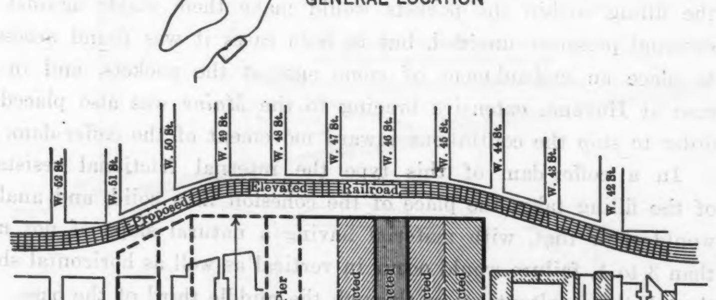
It was apparent from the general configuration of Manhattan Island in this vicinity that rock would be encountered over the inshore portion of the site of the proposed pier and slips. This rock surface was developed inshore from the existing bulkheads by wash-borings, and outshore from the bulkheads in the then existing slips by shod test piles. The development over the entire pier site out to the pierhead line, by these means and the usual test piles, indicated that the character of the river bottom varied from rock at an average depth of about 20 ft. below mean low water at the inshore end to a very soft mud at the outshore end, where it would be necessary to use fully lagged piles, from 85 to 89 ft. long.

In order to make proper depth of water for the giant liners, drawing, when loaded, about 38 ft., it was determined to provide for a depth in the slips of 44 ft. below mean low water, as records show that extreme low water has occurred 4 ft. below mean low water, and it was decided that there should be a clearance of at least 2 ft. between the ship's keel and the bottom of the slip, especially in that portion where rock forms the floor of the slip.

The sub-surface conditions thus developed and the decision to secure a depth of 44 ft. below mean low water in the slips plainly indicated the necessity for adopting a type of pier which would reduce to a minimum the quantity of submarine work to be done, and a method of procedure which would insure safe, workmanlike, and economical construction. Accordingly, a solid filled type was adopted for the inshore portion, about 220 ft. long, consisting of concrete retaining walls, to retain the filling and provide wharfage face, founded on bed-rock, thus utilizing the rock bottom over this portion of the pier area.




GENERAL LOCATION




DETAILED LAYOUT

WEST 46th STREET IMPROVEMENT
FOR LONG STEAMSHIP PIERS

FIG. 1.

The larger quantity of rock excavation necessary with submarine blasting over that within the coffer-dam, in order to make certain that the excavation would be carried to the lines and grades required; the advantage of being able to control the surfacing of the rock for the pier and slip walls, and to prepare the rock surface for the foundation of these walls; and the desirability of building these walls "in air"; all indicated the wisdom of prosecuting the work within a coffer-dam to be built so as to enclose the entire site of the work, extending from the center line of West 44th Street northward to the center line of West 47th Street, a distance of 780 ft.

## Design.

Having decided on the coffer-dam method of construction, the type to be used was made the subject of careful consideration. The area to be enclosed prohibited the use of bracing, and the unbraced steel sheet-pile type of coffer-dam was naturally suggested. Coffer-dams of this type had been used previously by the United States Government, at Black Rock and at Havana. There it was expected that the filling within the pockets would make them stable against the external pressure, unaided, but in both cases it was found necessary to place an embankment of stone against the pockets, and in the case at Havana, extensive bracing to the *Maine* was also placed, in order to stop the continuous inward movement of the coffer-dam.

In a coffer-dam of this type the internal frictional resistance of the filling takes the place of the cohesion in a solid, and analysis would show that, with material having a natural slope of not more than 3 to 1, failure would occur in vertical as well as horizontal shear, though the resultant passes through the middle third of the base.

The material overlying the rock at West 46th Street is a clayey mud, the natural slope of which is probably not as good as 3 to 1, thus prohibiting its use to obtain stability.

It had been decided to dredge over the site of the coffer-dam to 30 ft. below mean low water, leaving the remaining material to furnish support for the sheet-piles. The difficulty of removing the material from within the pockets and substituting better, as well as the greater cost of a coffer-dam of this type, due to the larger quantity of steel sheet-piling required, led to the adoption of the type shown by Fig. 2.

It should be noted that, where no embankment is placed against the pockets, the sheet-piles will be subjected to the pressure of the entire depth of filling at all times, and as the water after pumping may be expected to remain in these pockets for at least one-half their depth, if not more, as was the case at Havana, these sheet-piles will also be subjected to this hydrostatic pressure, necessitating a closer spacing of the diaphragms than where embankments are used.

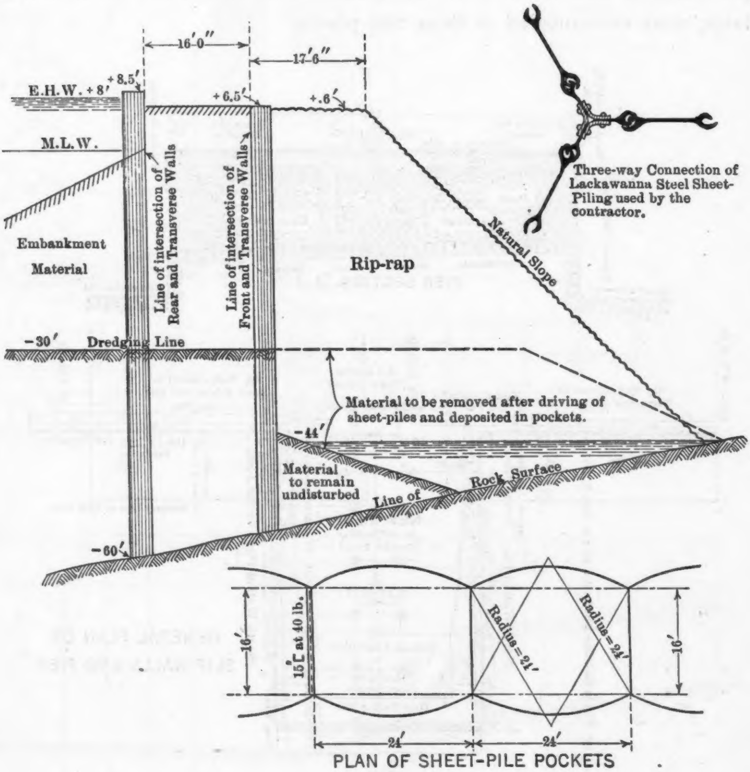



FIG. 2.

In the type of coffer-dam adopted, the sheet-pile pockets are used merely as a water seal, being supported against the external hydrostatic pressure by an embankment of rip-rap inside, and against the pressure of the rip-rap, during construction and before pumping, by a smaller embankment of earth outside.

This type of coffer-dam was adhered to, except along the crib bulkhead at West 47th Street, where, in place of the sheet-pile pockets, a single line of steel sheet-piling was driven, with an embankment of rip-rap in front, the space between the sheet-piles and the crib being filled with earth. Also, because of the limited space between the inner end of the West 44th Street Pier and the — 44-ft. rock contour line, two large cylinders filled with rip-rap were substituted for the pockets and embankment at this point. The only leaks, as described later, were encountered at these two places.

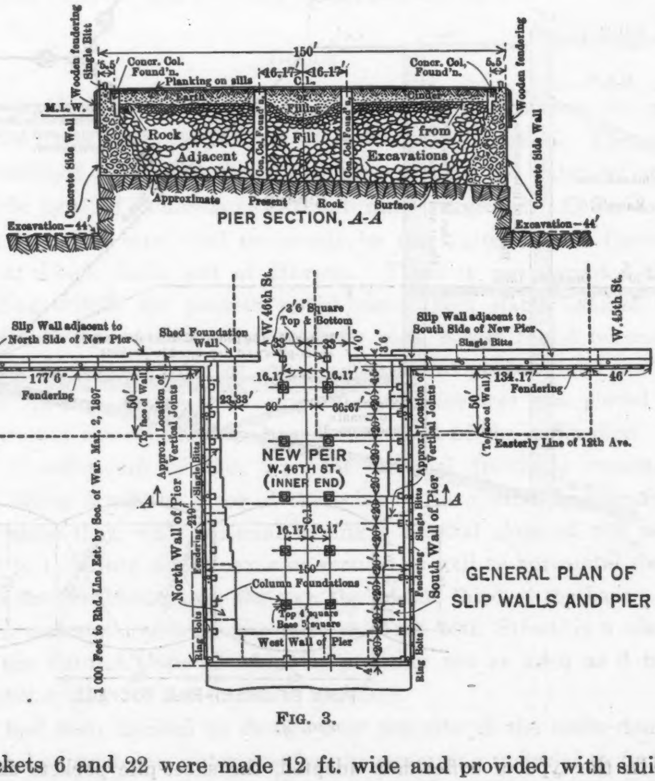
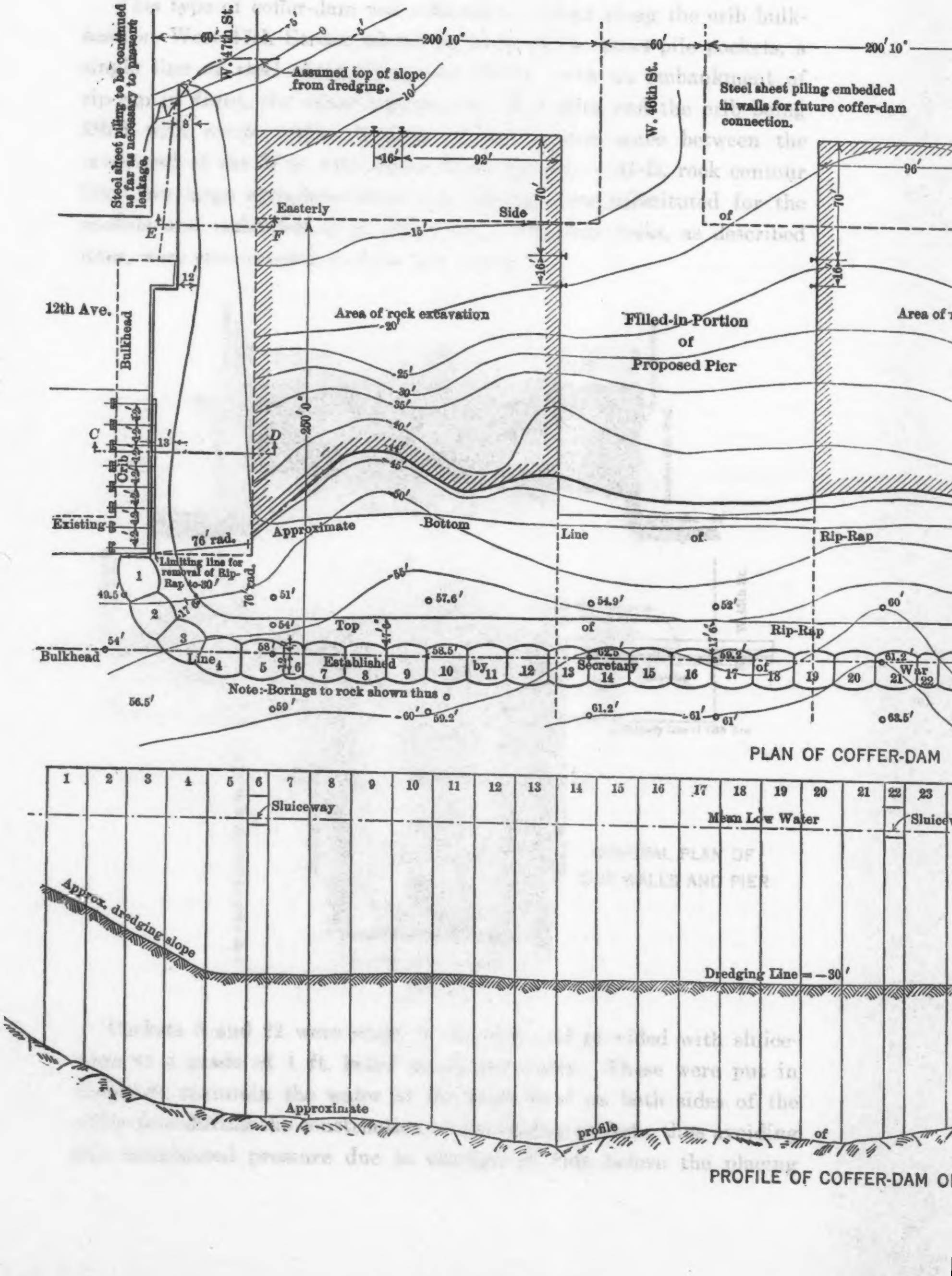



FIG. 3.

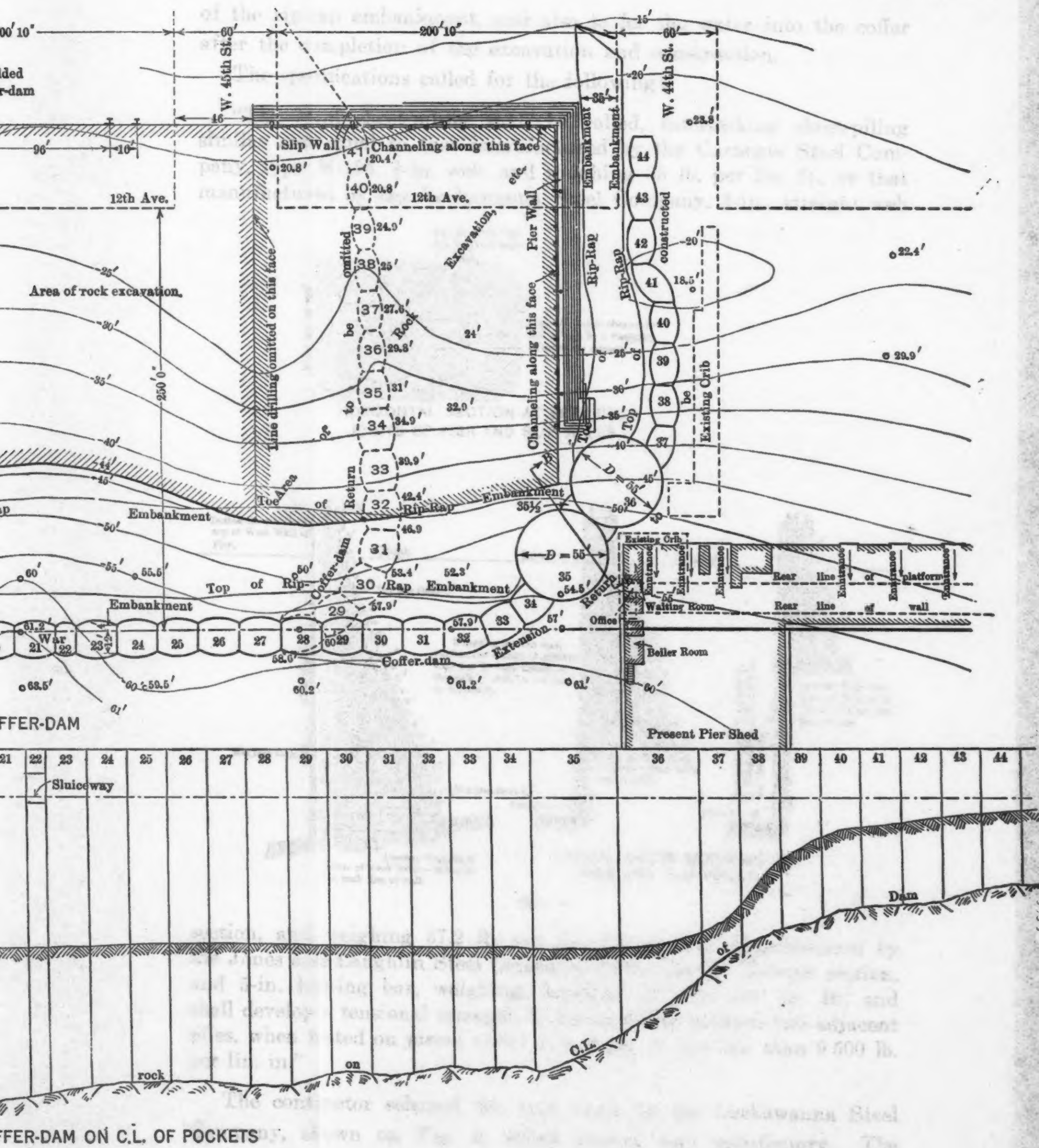
Pockets 6 and 22 were made 12 ft. wide and provided with sluiceways at a grade of 1 ft. below mean low water. These were put in chiefly to maintain the water at the same level on both sides of the coffer-dam during the construction of the closing pockets, thus avoiding any unbalanced pressure due to changes of tide before the placing




PLAN OF COFFER-DAM

PROFILE OF COFFER-DAM O

PLATE I.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
STANIFORD ON
UNUSUAL COFFER-DAM
FOR 1 000-FOOT PIER.



of the rip-rap embankment, and also to let the water into the coffer after the completion of the excavation and construction.

The specifications called for the following:

"The steel sheet-piling shall be rolled, interlocking sheet-piling similar and equal to that manufactured by the Carnegie Steel Company, type M-104, $\frac{3}{8}$-in. web, and weighing 38 lb. per lin. ft., or that manufactured by the Lackawanna Steel Company, $\frac{3}{8}$-in., straight web

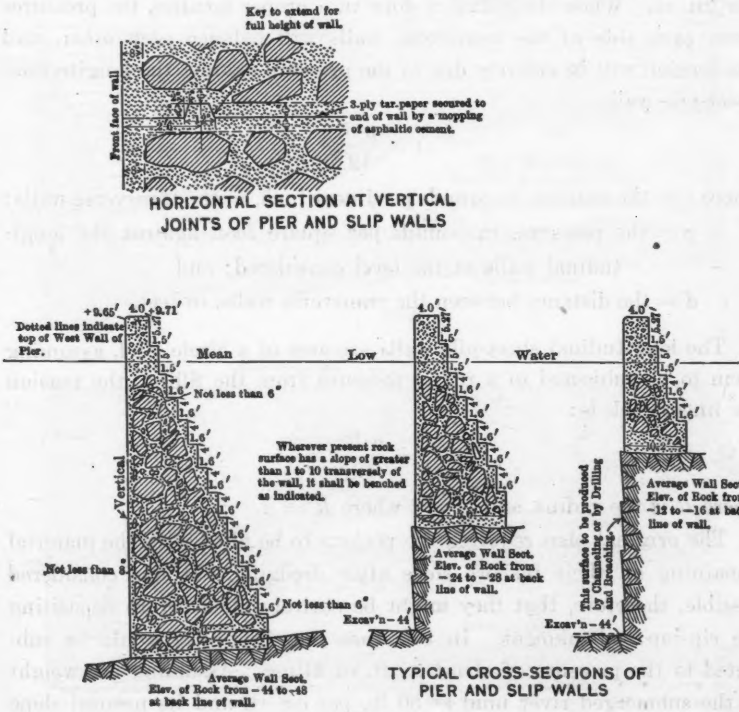



FIG. 4.

section, and weighing 37.2 lb. per lin. ft., or that manufactured by the Jones and Laughlin Steel Company, 12-in., special I-beam section, and 5-in. locking bar, weighing, together, 36.1 lb. per lin. ft., and shall develop a tensional strength in the interlock between two adjacent piles, when tested on pieces about 3 in. long, of not less than 9 500 lb. per lin. in."

The contractor selected the type made by the Lackawanna Steel Company, shown on Fig. 2, which proved very satisfactory. The

three-way connection pile used at the intersection of the transverse and outer walls of sheet-piling weighed about 85 lb. per ft.

### Stresses in Sheet-Pile Walls of Pockets.

Due to the pressure of the retained filling, the sheet-piles are all in tension transversely. Their resistance to this tension is limited by the strength of the interlock, which tests show will exceed 9 000 lb. per lin. in. When the filling is done in a proper manner, the pressures from each side of the transverse walls will balance each other, and the tension will be entirely due to the pressure against the longitudinal sheet-pile walls.

$$t = \frac{p\,d}{12};$$

where $t$ = the tension, in pounds per linear inch, in the transverse walls;
$p$ = the pressure, in pounds per square foot, against the longitudinal walls at the level considered; and
$d$ = the distance between the transverse walls, in feet.

The longitudinal sheet-pile walls are arcs of a circle, and, assuming them to be subjected to a radial pressure from the filling, the tension per linear inch is:

$$t_1 = \frac{p\,R}{12},$$

where $R$ is the radius, and $t_1 = t$ where $R = d$.

The original plan required the pockets to be filled with the material remaining on their inshore sides after dredging. It was considered possible, therefore, that they might be entirely filled before depositing the rip-rap embankment. In this case the sheet-piles would be subjected to the pressure of about 58 ft. of filling. Assuming the weight of the submerged river mud at 80 lb. per cu. ft. and its natural slope at 3½ to 1,

$$p = 0.57 \times 80 \times 58 = 2\,600 \text{ lb. per sq. ft.},$$

and as $r = d$, the maximum stresses in the longitudinal and transverse walls are equal and given by

$$t = \frac{2\,600 \times 24}{12} = 5\,200 \text{ lb. per lin. in.}$$

The possibility of avoiding unduly straining the sheet-piles by depositing some of the embankment before completely filling the

pockets was considered, and was actually found advisable, as explained later.

### Tests on Model Embankment.

To obtain information regarding the probable behavior of a rip-rap embankment under various conditions of pressure, some tests were made with a model, one-twentieth the size of the embankment, placed against the inshore side of the sheet-pile pockets, or 42 in. in height

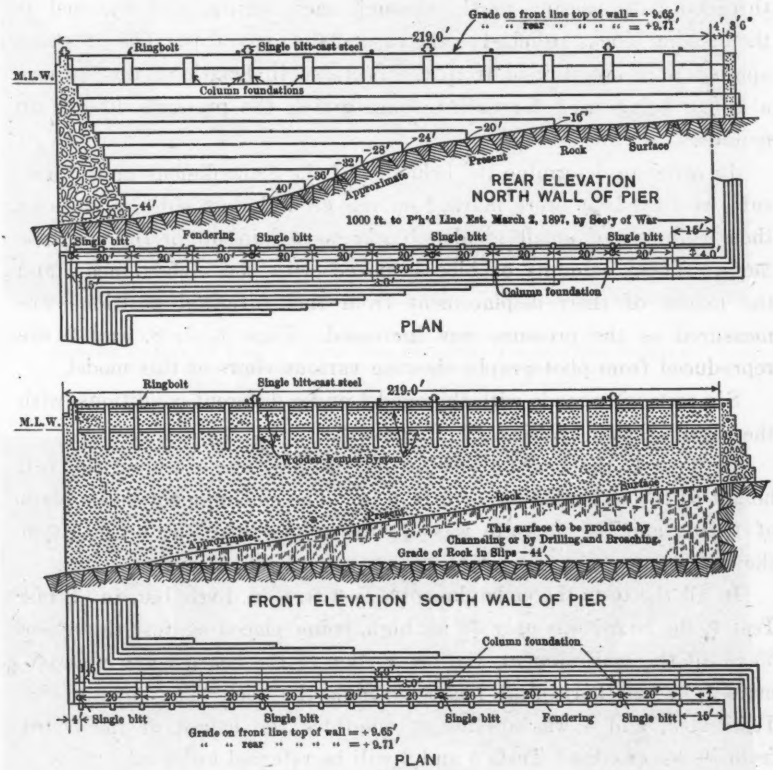



Fig. 5.

at the rear. These tests are incomplete, but, though it was found necessary to discontinue them, they are given here for whatever value may be derived from them.

The testing apparatus provided a bottom having the average slope of the rock floor at West 46th Street, 6 to 1, and also glass sides, 3 ft. apart, in order to observe the effect of pressures on the mass. The

board, against which the embankment was placed and by which the pressures were transmitted to the embankment, was fitted so as to slide inward on the bottom and between the glass sides. The embankment consisted of broken stone weighing as placed about 93 lb. per cu. ft., and varying from $\frac{1}{4}$ to $\frac{3}{4}$ in., which is about one-twentieth the size of rip-rap. The pressures were obtained by using two calibrated steel car springs, which were compressed by a nut working on a threaded bolt, passing partly through each spring, and fastened to the backing blocks attached to the base of the apparatus. The pressures applied were determined from the decrease in length of the springs, a gauge being used for each spring, giving the pressure directly in pounds.

In order to determine the behavior of the embankment under pressure, vertical lines were marked on the ground-glass sides, and along them columns of small wooden blocks were laid up in the embankment. These columns of blocks moved with the embankment, and the extent of their displacement from their original positions was measured as the pressure was increased. Figs. 6, 7, 8, and 9 are reproduced from photographs showing various views of this model.

Six tests were made with this model under different conditions, with the results shown by Figs. 11, 12, and 13.

In Test 1, the board transmitting the pressure extended the full height of the embankment, or 3 ft. 6 in. The embankment had a berm of 10 in., and the pressure was applied at one-third the height from the bottom.

In all the tests the embankment was 3 ft. 6 in. high, but, in all but Test 1, the board was only 33 in. high, being placed against the upper 33 in. of the embankment, the lower edge of the board being approximately in the horizontal plane through the toe. The pressure in Tests 2, 3, and 4 was applied at one-third the height of the board from its lower edge. Tests 5 and 6 will be referred to later.

The object of these tests was to determine the extent of the distortion which would take place in such an embankment under pressure, as well as its ultimate resistance.

Figs. 11 and 12 show the distortion throughout the mass in Tests 1 and 2, obtained by measurements of the displacement of the blocks of wood referred to. Also, the curves of deflection of the embankment with the increase of pressure are shown by Fig. 13.

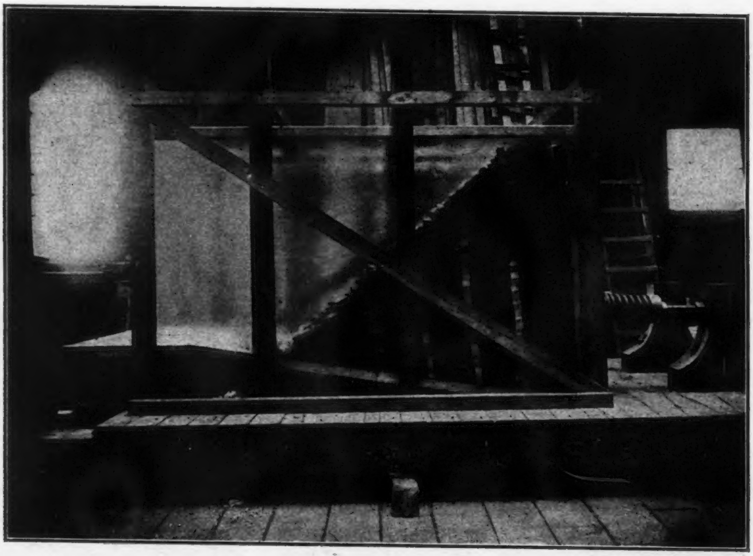


FIG. 6.—SIDE VIEW OF MODEL, SHOWING DISPLACEMENT OF COLUMNS OF WOODEN BLOCKS AT END OF TEST 1, WHEN THE PRESSURE AGAINST THE EMBANKMENT HAD REACHED 2 810 LB. THE BLACK VERTICAL LINES SHOW THE ORIGINAL POSITION OF THE COLUMNS OF WOODEN BLOCKS.

FIG. 7.—FRONT VIEW OF MODEL, SHOWING SLOPE OF BROKEN STONE EMBANKMENT.

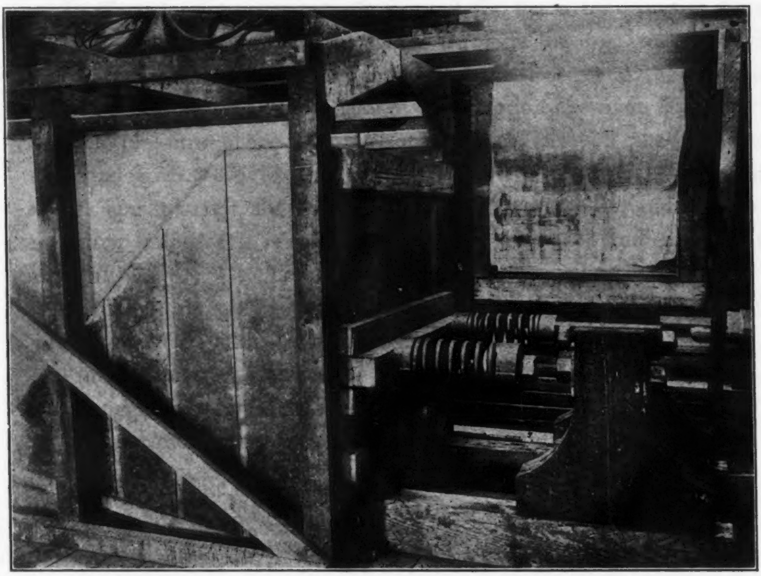


FIG. 8.—SIDE AND REAR OF MODEL, BEFORE APPLICATION OF PRESSURE IN TEST 2, SHOWING COLUMNS OF WOODEN BLOCKS LAID UP ALONG BLACK LINES ON GLASS SIDES.

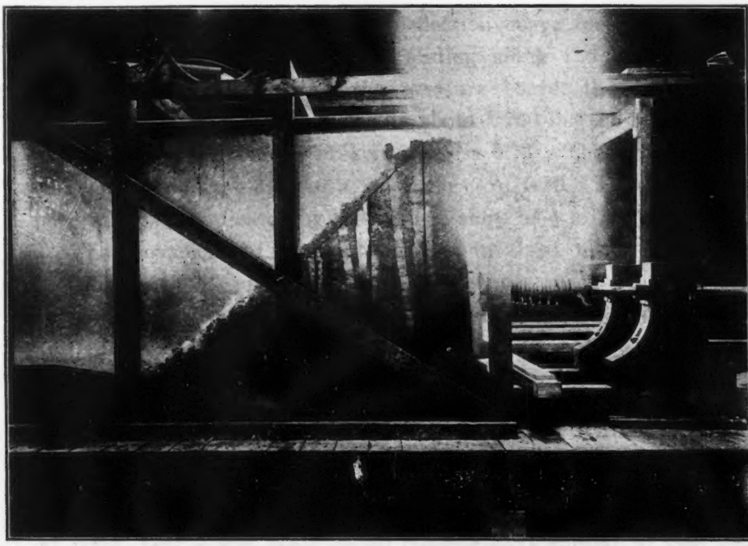


FIG. 9.—SIDE VIEW OF MODEL AT END OF TEST 2, WHEN PRESSURE HAD REACHED 1 810 LB. NOTE DISPLACEMENT OF WOODEN BLOCKS.

In Test 1, at a pressure of about 2 375 lb., a tendency to break, about through the horizontal plane through the toe, was observed, and is indicated by the breaks in the lines in Fig. 11 at this pressure, as well as in the break in the curve of deflection shown in Fig. 13.

The pressure above the horizontal plane through the toe corresponding with 2 375 lb. on the whole board is 1 460 lb. The weight of the embankment above this plane is about 1 900 lb., and the corresponding value of $\mu$ is 0.77, as compared with 0.83 indicated by the natural slope of $\frac{1}{2}$ to 1.

A second break occurs in the deflection curve in Fig. 13 at a pressure of 2 750 lb. Assuming this as an indication of sliding up the inclined bottom, taking the weight of the embankment at 2 380 lb., the tangent of the angle made by the bottom with the horizontal at $\frac{1}{8}$, and substituting those values in the formula,

$$\mu = \frac{P - \tan. \alpha\ W}{W + \tan. \alpha\ P},$$

we have a value for $\mu$, the coefficient of friction of the stone on the wood bottom, $= 0.83$. This high value is presumed to be due to the probability of the stones becoming embedded in the wood to some extent.

Considering the break in the deflection curve for Test 2, Fig. 13, at 1 280 lb. as an indication of sliding along the horizontal plane through the lower edge of the pressure board, the weight of the embankment above this plane being about 1 750 lb., the resulting value of $\mu$ for broken stone on broken stone is 0.73. The critical pressure of 1 000 lb. in Test 3 gives a value for $\mu$ of 0.80. These values compare with 0.83 indicated by the natural slope of $\frac{1}{2}$ to 1.

The movement of the embankment in these tests, accompanying each increase of pressure, would apparently cease in a very short time. The readings of both pressures and movement were then taken, and the next increase of pressure effected. It became evident, however, that, particularly for the higher pressures, further movement will occur when more time is allowed. This is illustrated in Test 2, Fig. 12, under pressure 8, in which case the pressure against the embankment dropped from 1 805 to 1 565 lb. over night, accompanied by a corresponding movement of the embankment. To determine the effect of time, Test 4 was made on an embankment similar to that used in

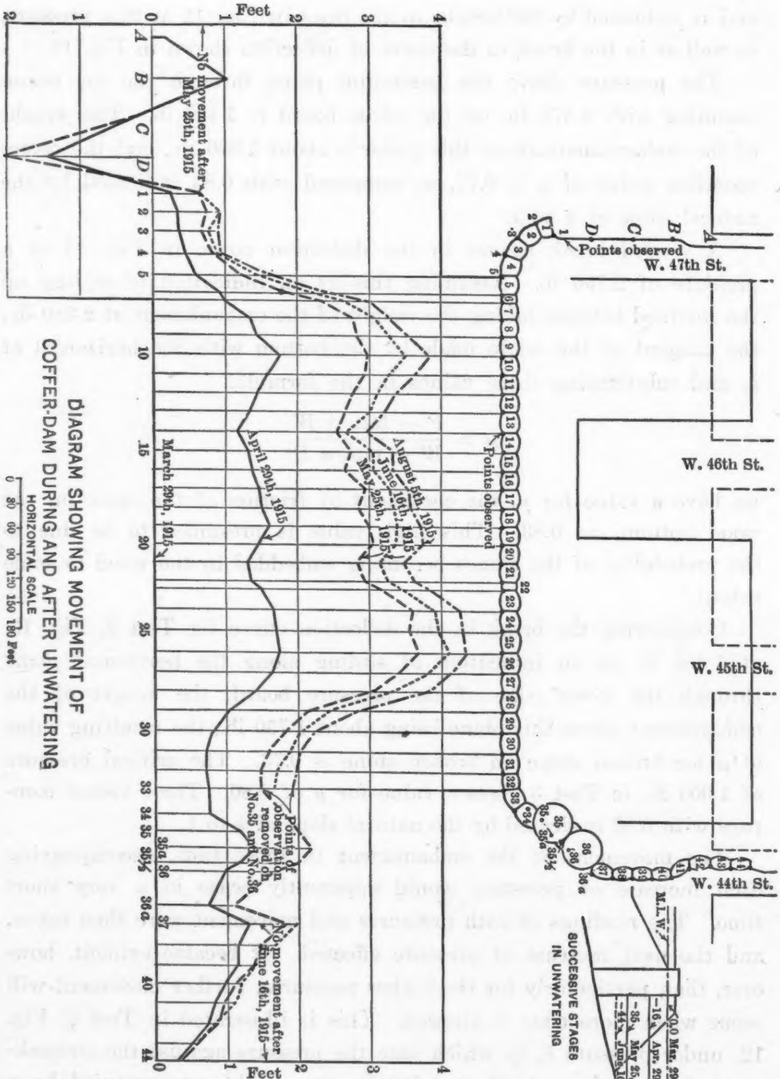



DIAGRAM SHOWING MOVEMENT OF COFFER-DAM DURING AND AFTER UNWATERING

FIG. 10.

Test 2, with a berm of 10 in. The springs were compressed until they indicated a pressure of 1 060 lb. The accompanying movement of the embankment at the top was 0.07 in. At the end of 1 week, the springs indicated a pressure of 950 lb., and this drop in pressure was accompanied by an additional movement of 1.045 in. at the top of the embankment. At the end of 1 month, the pressure had dropped to 900 lb., with a further displacement of the embankment at the top of 0.08 in. Observation of the embankment at the end of the second month indicated no further change either in pressure or displacement.

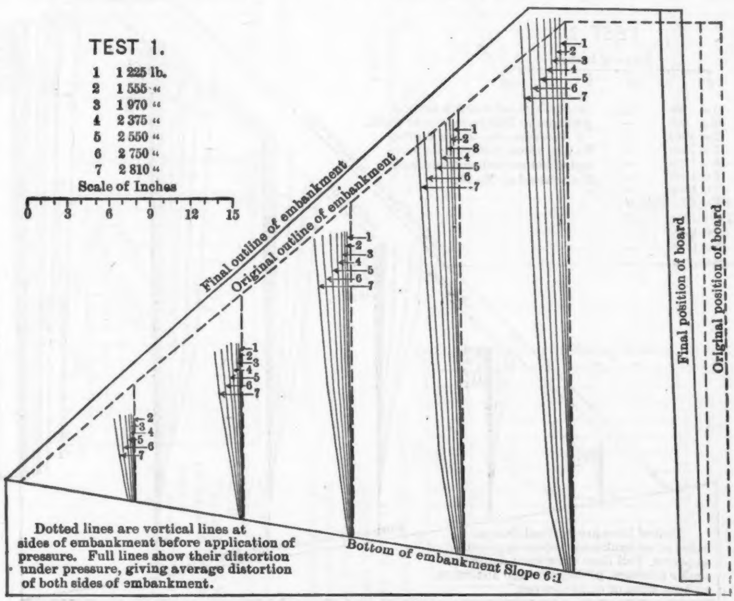



FIG. 11.

This test does not show that the embankment would not have come to rest in time under a pressure greater than 900 lb. constantly applied, which could not be determined with the apparatus used, as any movement of the embankment necessarily results in a partial release of the springs, and consequently a decrease in pressure.

Tests 5 and 6 were undertaken to determine the effect of applying the pressure at a point above the center of gravity of the embankment as well as the effect of friction along the face of the embankment in contact with the board.

The embankment used in these tests was the same as that in Test 2, except that the width of the berm was about 6 in. In the case of Tests 5 and 6, the pressure was applied at two-thirds of the height above the bottom of the board. In Test 5 the board was free to move, as in the previous tests, and in Test 6 the board was anchored at the bottom with hinges. The results are given by the curves of deflection of the top and bottom of the board, as shown in Fig. 13.

The curve for Test 5 indicates an ultimate resistance of about 600 lb. to a pressure applied at two-thirds of the height, which about

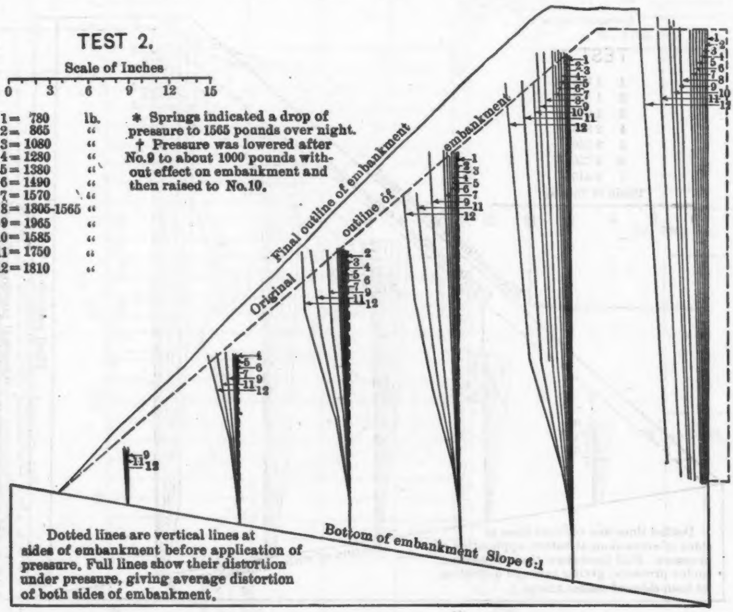



FIG. 12.

agrees with the theoretical resistance, on the assumption of a coefficient of friction of 0.83 developed along horizontal planes in the mass. This is only one-half the resistance of the similar embankment used in Test 2 under pressure applied at one-third of the height.

The tendency of the rear of the embankment to rise under pressure, carrying the board with it, was very evident in Tests 1, 2, and 3. In Tests 5 and 6 the movement was entirely horizontal. To determine the effect of friction between the board and the face of the embankment, the board, in Test 6, was anchored as mentioned pre-

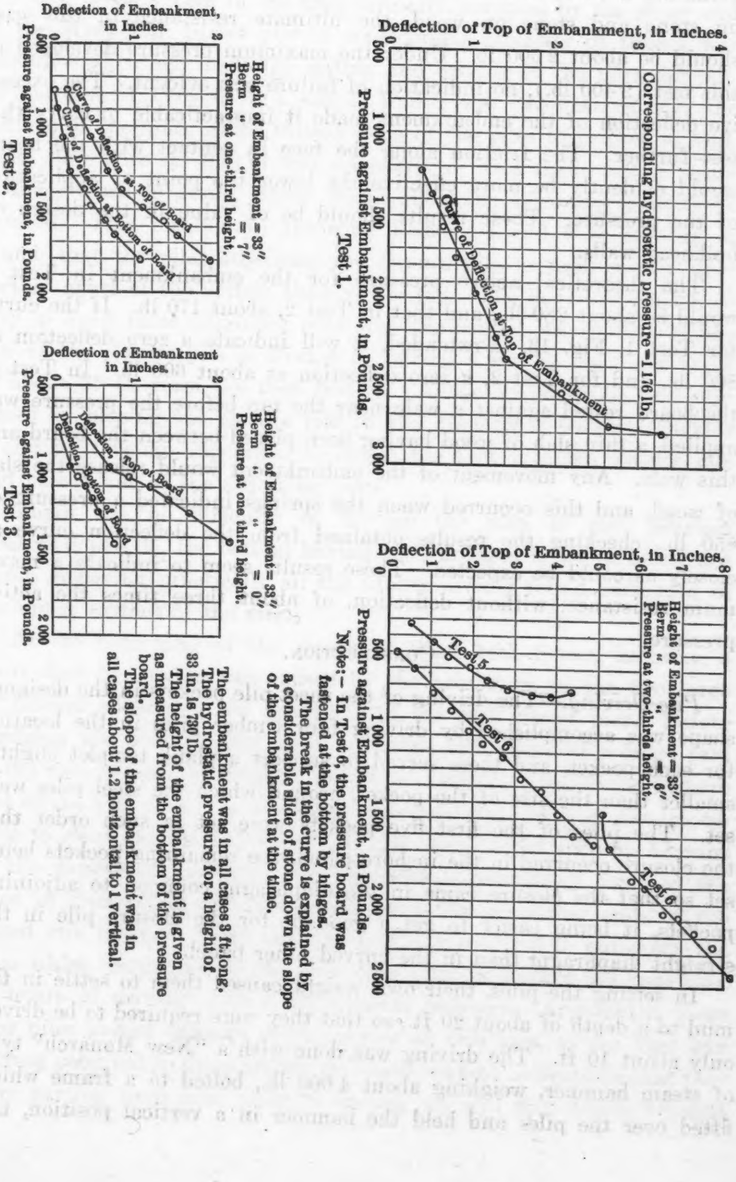

Deflection of Top of Embankment, in Inches.
Corresponding hydrostatic pressure = 1 176 lb.
Curve of Deflection at Top of Embankment
Pressure against Embankment, in Pounds.
Test 1.
Deflection of Top of Embankment, in Inches.
Height of Embankment = 33″
Berm " = 6″
Pressure at two-thirds height.
Test 5
Test 6
Test 6
Pressure against Embankment, in Pounds.
Note:— In Test 6, the pressure board was fastened at the bottom by hinges.
The break in the curve is explained by a considerable slide of stone down the slope of the embankment at the time.
Deflection of Embankment, in Inches.
Height of Embankment = 33″
Berm " = 7″
Pressure at one-third height.
Curve of Deflection at Top of Board
Curve of Deflection at Bottom of Board
Pressure against Embankment, in Pounds.
Test 2.
Deflection of Embankment in Inches.
Height of Embankment = 33″
Berm " = 0″
Pressure at one third height.
Deflection, Top of Board
Deflection, Bottom of Board
Pressure against Embankment, in Pounds.
Test 3.
The embankment was in all cases 3 ft long.
The hydrostatic pressure for a height of 33 in. is 730 lb.
The height of the embankment is given as measured from the bottom of the pressure board.
The slope of the embankment was in all cases about 1.2 horizontal to 1 vertical.

FIG. 13.

viously. The result is shown by the lower curve, Fig. 13. Considering the resistance developed along horizontal planes in the embankment, and assuming a value of 0.83 for the coefficient of friction for both stone on stone and stone on wood, the ultimate resistance in this case should be about 2 500 lb. Under the maximum pressure developed in this test (2 400 lb.), no indication of failure was evident. The excessive deflection of the embankment made it impracticable to carry this test further. The friction along the face in contact with the board would evidently be more effective the lower the point of application of the pressure. These results should be of value in the design of bulkhead walls.

The theoretical active pressure for the embankment in Test 1 would be about 280 lb., and that in Test 2, about 170 lb. If the curve for Test 1, Fig. 13, is extended, it will indicate a zero deflection at 800 lb., and for Test 2, a zero deflection at about 600 lb. In Test 4, the board rested against a wale near the top before the pressure was applied, a thin slab of wood having been placed between the board and this wale. Any movement of the embankment would release the slab of wood, and this occurred when the springs indicated a pressure of 850 lb., checking the results obtained from the deflection curve as closely as could be expected. These results seem to indicate a maximum resistance, without deflection, of about three times the active pressure.

## Construction.

*Pile-Driving.*—The driving of the sheet-pile pockets in the designed shape was accomplished by driving four timber piles in the location for each pocket, and these served to support a plank templet slightly smaller than the size of the pocket, around which the steel piles were set. The piles of the first five pockets were set in such order that the closure occurred in the inshore panel, the remaining pockets being set so that the closure came in the diaphragm common to adjoining pockets, it being easier to get a good fit for the closure pile in the straight diaphragm than in the curved inner panel.

In setting the piles, their own weight caused them to settle in the mud to a depth of about 20 ft., so that they were required to be driven only about 10 ft. The driving was done with a "New Monarch" type of steam hammer, weighing about 4 000 lb., bolted to a frame which fitted over the piles and held the hammer in a vertical position, the

hammer plate being wide enough to drive three piles at once. After being driven in sets of three, the individual piles were driven with a follower, in order to insure contact of each with the rock. As the first five pockets were in the area formerly occupied by the inner end of the West 47th Street pier, a considerable quantity of timber was encountered, which made the setting and driving of the piles difficult. In the case of Pockets 3, 4, and 5, timber was encountered near the surface of the mud, which caused some of the piles to project so high when they were set that it was necessary to drive them with a steam hammer down to grade, which would allow the adjoining piles to be entered. This resulted in throwing Pockets 4 and 5 so far out of plumb that the piles adjoining the closure pile in each pocket were farther apart at the bottom than at the top. The hard driving necessary to draw these adjoining piles together caused the rupture of one jaw of the closure pile in Pocket 5, leaving a gap in the inner panel of about 25 ft. from the rock bottom. A similar condition was produced in placing the closure pile in the diaphragm between Pockets 8 and 9, it being impossible to detect the fact that the closure pile had passed out of the adjoining jaws.

There was no difficulty in driving the piles to rock between Pockets 8 and 35, the bottom having been dredged to 30 ft. below low water, removing practically all the timber usually found embedded in the upper surface of the mud.

At the beginning of the return at West 44th Street, however, a great deal of trouble was caused by the proximity of the two large cylindrical pockets to the inner end of the 44th Street pier. Before dredging was done at this point, a timber crib occupied a large part of the area covered by Pockets 35 and 36, extending under the inshore end of the pier shed. In dredging with a dipper-dredge it was impossible to clear out the bottom of the crib close to the shed on account of the danger of damaging the structure, consequently some of the piles in Pockets 36 and 35½ encountered the projecting ends of the round crib timbers. Very hard driving was required to put down the piles which were close to the crib, four piles in Pocket 35½ requiring 24 hours of driving with a 2-ton steam hammer. Apparently, these four piles passed through three courses of crib logs, as the very bad driving occurred in three stages, yet there were no indications that they were parting from each other or curling up, and they finally brought

up solidly on the rock. Evidently, however, there was a rupture, as practically all the leakage of the entire dam took place through the outshore side of Pocket 35½.

The setting of pockets, which was begun with No. 1, on April 10th, 1914, was continued until No. 27 was reached, on June 19th, 1914. Owing to a change in plan, setting was then suspended until October 5th, when it was resumed in No. 35, the cylindrical pocket, and carried thence to Pocket 44. Pockets 28 to 30 were then set, Nos. 31 to 33 being omitted until the lighter had completed the removal, from the rear of the dam, of all the mud which it could not reach from a position outside the dam. The lighter was then placed outside the dam and the remaining pockets were set, the closure being made in Pocket 31 in the middle of the inshore and outshore panels.

*Dredging and Filling Pockets.*—The weakness at these two points was soon developed when the filling of the pockets was begun. In the case of Pocket 5, the mud had been dredged from the inshore side to a depth of about 30 ft. below low water, the material being placed in the pocket to a grade of about 7 ft. above low water. There was thus a head of about 37 ft. of mud on the inner panel, which, after the lapse of a few hours, completed the opening of the gap at the closure pile.

In order to repair the damage it was necessary to remove some of the filling which had been placed in Pockets 6 and 7, and thus allow the corner pile between Pockets 5 and 6 to be pulled back into a vertical position. The distorted piles of the ruptured panel were removed and new piles were placed, two being added to the original number to fill the space caused by the spreading of the corners when the filling was suddenly released.

The failure of the diaphragm between Pockets 8 and 9 occurred when the filling in No. 8 had reached — 12, and in No. 9 when it had reached — 20, and the mud inshore from the two pockets had been dredged to about — 40 ft. The original plan, in constructing the dam, was to dredge the material in the rear overlying the rock, place it in the pockets, and then proceed to deposit rip-rap at the rear; and, when the rip-rap had reached the level of the mud, outshore from the dam, to deposit filling outshore in sufficient quantity to counteract the pressure of the rip-rap.

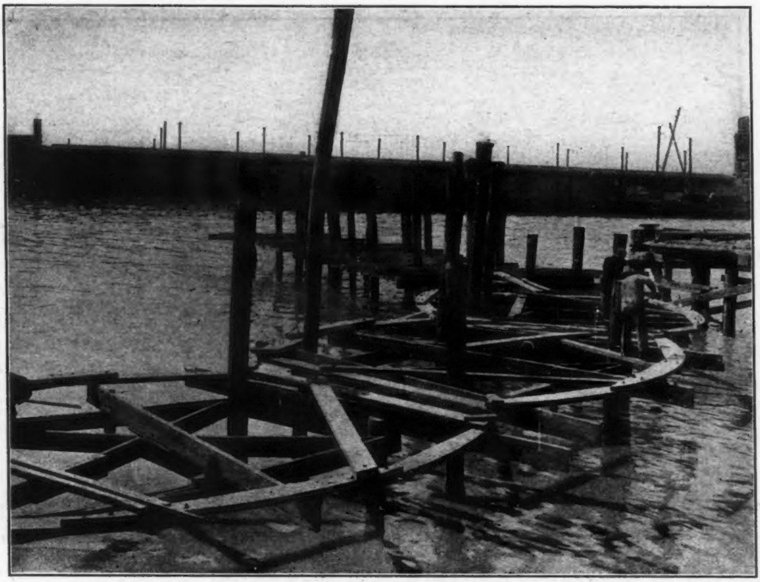


FIG. 14.—LAYING OUT TEMPLATES FOR DRIVING SHEET-PILING.

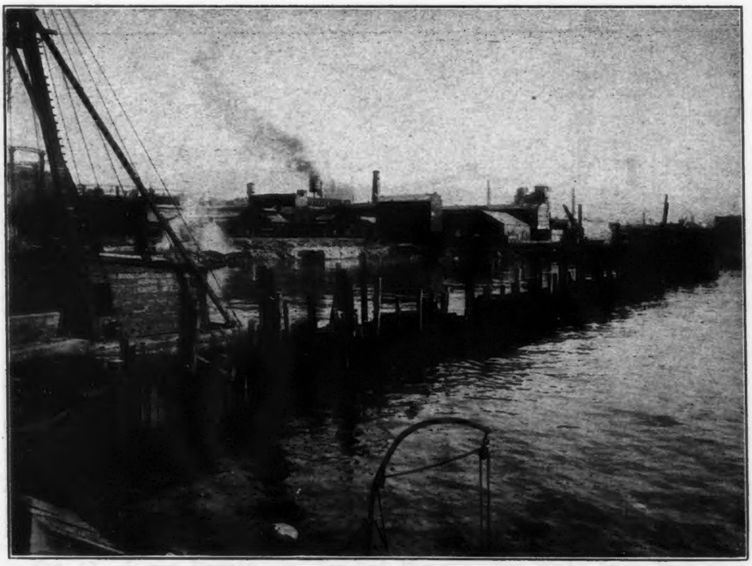


FIG. 15.—SHEET-PILING FOR POCKETS DRIVEN.

FIG. 16.—DRIVING STEEL SHEET-PILING FOR COFFER-DAM.

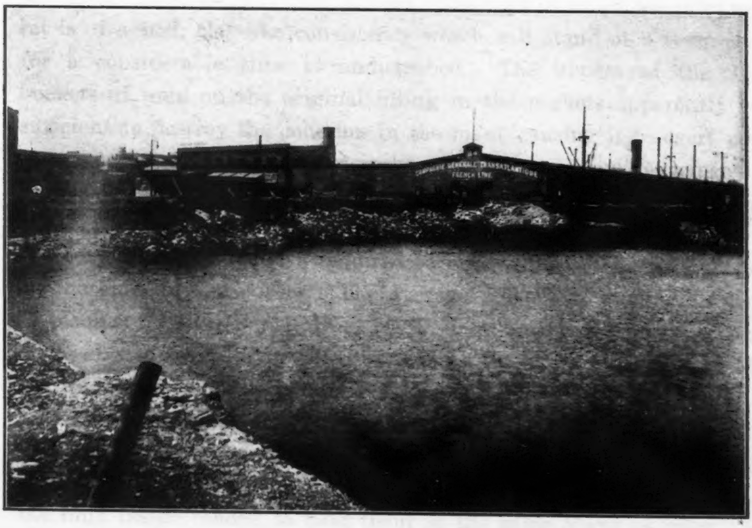



FIG. 17.—LARGE POCKET, AND RIP-RAP EMBANKMENT, WEST 44TH STREET.

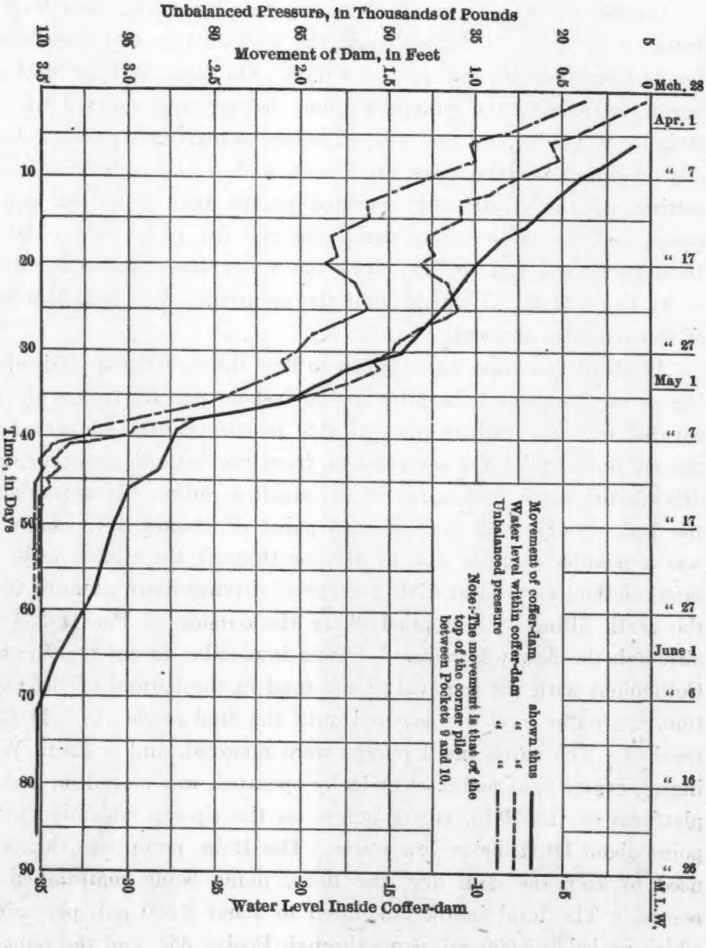
The failure of Pockets 5 and 8 made it evident that this procedure would have to be modified, unless there was assurance that the closure piles were not under considerable initial stress. A number of the closure piles had required considerable driving, and it was decided to avoid unduly stressing them by proceeding as follows: The mud in the rear of the dam was excavated for the length of two pockets and taken to sea; rip-rap was then deposited until it reached the grade of the original bottom in the pockets; filling was then placed in the pockets, and a head of mud and earth of about 10 ft. over the grade of the rip-rap was maintained as the placing of rip-rap was continued. Sufficient tension was thus maintained in the walls of the pocket to make them water-tight without subjecting them to severe loads. The filling comprised a mixture of mud and cellar earth, the latter being dumped from trucks which were driven along a platform built on top of the dam. As the earth was dumped in the middle of the pockets, its greater weight forced the mud up along the walls, providing a seal which proved to be absolutely water-tight.

During the time required for the foregoing operation of dredging and filling, which averaged about 10 days for each pocket, the unbalanced pressure of the mud in the pockets and also that outshore from the latter caused them to lean inshore. The mud in which the piles were set is of a stiff, clay-like consistency which will stand at a steep slope for a considerable time if undisturbed. The impact of the 2-yd. buckets of mud on the original filling in the pockets apparently was sufficient to destroy the cohesion in the mass, causing it to exert pressure tending to tip the pocket inshore. The maximum movement of a pocket (No. 16) immediately after the completion of the dredging was ½ ft. in 24 hours, and there was a maximum total movement of about 2½ ft. in the 10 days required to restore the balance of pressure on the sides of the pocket. The average movement during the time of placing the filling was about 1½ ft. The balance of pressure, once restored, was easily maintained by placing the earth embankment outshore to counteract the pressure of the rip-rap.

In dredging and filling the cylindrical pockets (Nos. 35 and 36) a different procedure was necessary, on account of the change in the type of construction. Whereas, in the small pockets, the pressure of the mud inside tended to hold them in the same shape as when they were set, in these cylinders, as it was necessary to clear the inside

of mud in order to bring the rip-rap in contact with the rock bottom, there was some danger that the pressure of the mud around the outside might distort or collapse the cylinders. This danger was avoided by dredging around the outside of the cylinders at the same time that deepening was going on within, except on the sides which were adjacent to the crib. Here the proximity of the crib precluded any dredging, and it was necessary, in order to hold the pockets in shape, to leave a fillet of stiff mud around that portion of the circumference which received pressure from the outside. In each cylinder the rock was cleared of mud for about three-quarters of the area of the bottom. Immediately after the completion of dredging the rip-rap was deposited uniformly around the circumference, maintaining practically the circular shape throughout the operation.

*Pumping.*—The unwatering of the coffer-dam was begun on March 29th, 1915, with a Morris 10-in. centrifugal steam pump, set on skids resting on the rip-rap embankment. Steam for this pump was furnished by a 50-h.p. locomotive boiler placed on the top of the dam. With this pump the level of the water was lowered from 0 to — 13 ft. in 19 days, the pump being operated only 16 hours per day, in order to allow the rip-rap embankment to consolidate itself. Meanwhile (April 13th), a leak began developing in the single line of piling along the face of the crib at 47th Street, about 100 ft. from the river, and, by April 18th, this had reached the capacity of the pump. The single line of piling which forms the dam along 47th Street was driven on a line parallel to, and from 4 to 5 ft. from a rock-filed timber crib which extends along 47th Street. Along the face of this crib, at the point where the leakage took place, there was none of the stiff mud which acted as a seal elsewhere along the dam. Furthermore, the rock fill in the cells of the crib was so porous that the river water had free access to the piling, and, although earth filling was deposited between the face of the crib and the sheet-piling, its thickness (about 4 ft.) was not sufficient to prevent a considerable flow between the piles and the rock. In order to place against the foot of the piling material which would not be drawn through by the flow of water, wooden piles were driven at intervals of from 4 to 5 ft. between the steel piling and the face of the crib. These were then withdrawn, and the holes filled with a mixture of sand and horse manure by ramming down charges of this material with the pile until the stiffness




MOVEMENT OF COFFER-DAM FROM BEGINNING OF PUMPING.

FIG. 18.

of the driving indicated that the cavity was filled. By thus introducing material which would clog easily, and by consolidating the earth filling for about 50 ft. along the sheet-piling, the flow was reduced to small proportions.

During the 9 days required to stop this leak, the water level rose from — 13 to — 11 ft., although the 10-in. pump had been running for 24 hours per day for the last 4 days. On April 22d the level again began to recede, the pumping plant being supplemented on April 29th by two 6-in. and one 4-in. Alberger centrifugal pumps, electrically operated. These were on a raft within the coffer-dam. After setting up the electrically operated pumps they were run continuously, and the 10-in. steam pump was run for 16 hours per day. In 16 days (April 22d to May 8th) the water level was reduced from — 11 to — 34 ft., at which point the accumulated mud in the bottom of the area was exposed.

At about this time water began to flow through Pocket 35½, appearing at the bottom of the piles in the inshore panel. It was then discovered that the closure piles of this inshore panel had parted from the adjoining piles for several feet from the bottom, probably due to driving the panel in a curve of too small a radius. It appeared that the leak occurred in the outshore panel of Pocket 35½, where there was a possible rupture due to driving through the timber crib. The same method as used at 47th Street—of driving down manure to hold the earth filling—was applied along the outside of Pocket 35½, and, although the flow was reduced, it was impossible to cut it off entirely. Coincident with the removal of the mud in the bottom of the excavation, the water level was lowered until the final grade of — 44 ft. was reached. The three small pumps were removed, and a 12-in. Worthington centrifugal pump, electrically operated, was placed on the same platform as the 10-in. steam pump, on the rip-rap embankment at a point about 30 ft. below low water. The 12-in. pump was thenceforth used to keep the dam dry, the 10-in. pump being maintained as a reserve. The total inflow amounted to about 2 500 gal. per min., of which probably 2 000 gal. came through Pocket 35½, and the remainder from the cut-off along 47th Street and from the upland. There was no indication of any leakage whatever through the portion of the dam composed of the small cells, and it is evident that, had conditions permitted their use throughout, the leakage would have been slight.

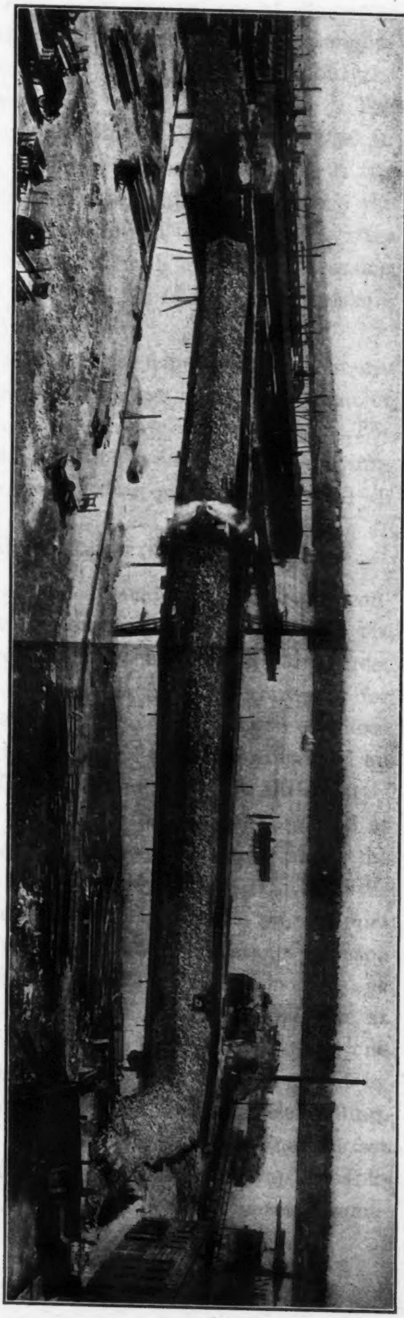


FIG. 19.—LEVEL OF WATER INSIDE COFFER-DAM, MAY 1ST, 1915, 17.9 FT.

FIG. 20.—CONCRETE CORNERS AND LOWER COURSE FOR RETAINING WALL, SEPTEMBER 10TH, 1915.

During the progress of unwatering, daily observations of the movement of the dam were made, so that, if necessary, the rate of pumping might be changed in order to check undue movements, but the changes were at no time so great as to cause apprehension. The maximum inshore movement at any one point in 24 hours was 0.40 ft. at Pocket 25, when the water level was lowered from — 24 to — 27 ft.; and the total movement during the 40 days required to remove the water to — 34 ft. was $2\frac{1}{2}$ ft. The dam continued to lean inshore after pumping had been completed to — 44 ft., with a slowly decreasing rate, coming practically to rest by June 15th, 1915. The total inshore movement of the top was about $3\frac{1}{2}$ ft.

Fig. 10 shows the alignment of the coffer-dam for several stages of the unwatering, and Fig. 18 shows the history of the movement of a typical section of the coffer-dam as the unwatering proceeded. The unbalanced pressures were calculated on the assumption of a hydrostatic head from rock at the inner row of sheet-piles at — 60 to + 4.

## Plant, and Rock Removal.

The plant provided for handling the excavated material and concrete comprised eight 15-ton steam locomotive cranes, of which six were used in the slip and pier areas and two on the old 45th Street pier. These narrow-gauge steam locomotives provided transport for concrete and spoil. The six cranes used in the coffer-dam were placed so that all parts of the area could be covered, three being carried on timber trestles extending inshore from the dam, the other three partly on trestles and partly on the two pier side-walls and the slip wall at 44th Street. There were two lines of narrow-gauge track along the top of the dam extending out on the old 45th Street pier, where the two cranes loaded the rock into scows lying alongside the pier. Spurs extended to the bulkhead along 47th Street and to the cranes on each trestle. There were also three stiff-leg derricks with 70-ft. booms, one at 47th Street, about 50 ft. outshore from the bulkhead wall, one at the north side of the pier, in the rear of the bulkhead wall, and one near 44th Street, at the bulkhead. These derricks were operated by compressed air, supplied by the plant provided for the rock drilling.

In placing the trestles for the locomotive cranes, a crane was set up on top of the dam, and the mud at the toe of the rip-rap embankment was excavated with a $1\frac{1}{2}$-yd. "battleship" which operated some-

what after the manner of a clam-shell bucket. As the toe of the slope was beyond the reach of the crane boom, several 24-in. I-beams were laid on the rip-rap, and the bucket was allowed to slide down them. After the rock had been uncovered, a trestle bent was erected at the foot of the bank, and an intermediate bent half-way up the slope. These bents were spanned by five 24-in. I-beams on which the crane track was laid. Successive bents were erected as the rock excavation proceeded, the trestles finally being extended so that the cranes would reach the bulkhead.

The rock excavation was begun at the toe of the rip-rap bank, and carried thence toward the bulkhead. Ingersoll-Rand tripod air drills were used until the depth of the cut was about 22 ft., and then Cyclone well-drilling machines, operated by gasoline engines, were substituted. The small drill holes, which were started with a 3½-in. and finished with a 1½-in. bit, were about 4 ft. from center to center. The well drill holes were 6 in. in diameter, and were about 14 ft. from center to center. Along the line of the pier and bulkhead walls, line drilling was done with the tripod drills, the holes being from 6 to 8 in. apart.

There was considerable difficulty with some of the small drill work, on account of the seamy nature of a part of the rock; this was avoided by the use of the well drills. The rock in this locality is composed of hard gray granite and Manhattan schist, with little order or arrangement in their occurrence. In the outshore portion of the excavation, the rock proved to be a fine, sound granite, which furnished a good foundation for the pier side-walls, but, in the area east of a line about 75 ft. from the bulkhead, granite and schist were encountered in irregular layers. The strata of the schist stood nearly on edge, as is usual, and varied from a hard to a quite soft, much laminated, structure. The granite, particularly within the area of the bulkhead wall, was badly broken up by seams, running in all directions and mostly filled with mud. In a number of locations the rock was of such poor character that it was necessary to excavate within the wall area, down to the grade of — 44 ft., in order to provide proper support for the front of the wall.

The rock was disposed of by loading it by hand into steel "battleships" and wooden skips, which were hoisted by the locomotive cranes to flat cars of about 3 cu. yd. capacity. These cars were transported

FIG. 21.—DRILLING, BLASTING, AND REMOVING ROCK, JULY 9TH, 1915.

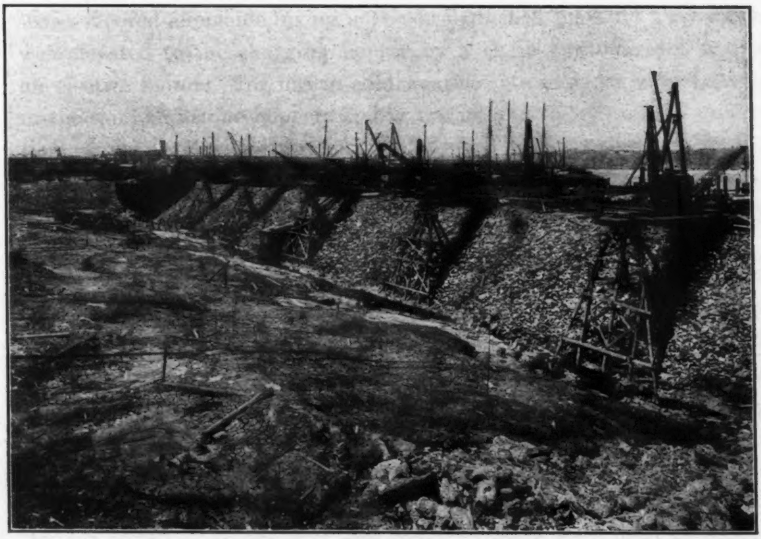


FIG. 22.—WITHIN THE COFFER-DAM, JULY 9TH, 1915.

by locomotive to the pier at West 45th Street in trains of from six to eight cars, and there unloaded by two locomotive cranes into bottom-dumper and deck scows. The rock thus loaded was shipped to Riverside Park, between West 88th and West 95th Streets, and there deposited along the shore in a trench, so as to form an embankment for the extension of Riverside Park. Other portions of the rock excavated were used for back-filling in the rear of the pier and bulkhead walls, and for the cyclopean concrete.

## Pier and Slip Walls.

The plant for the concrete work was erected on a pile platform, about 30 ft. wide and 100 ft. long, built adjoining the outshore side of the dam near the 47th Street end. On the platform there was a cement shed with a capacity of 4 000 bags. A 1-yd. Ransome mixer, electrically operated, was placed at an elevation which allowed it to discharge into buckets set on flat cars on the track running along the top of the dam. Hoppers for stone and sand, of 90 and 45 cu. yd. capacity, respectively, were erected directly over the mixer, resting partly on the dam and partly on the platform. A timber stiff-leg derrick, with a 70-ft. boom, operated by compressed air, and set at the same elevation as the hoppers, supplied them with sand and stone from scows moored alongside, using a 1½-yd. clam-shell bucket. The cement was elevated to the charging hopper by a chain bag conveyor run by an electric motor. For use in cold weather, steam pipes were inserted in the sand and stone hoppers and in the mixing water tank. In operation, a flat car with two bottom-dumping buckets, each holding a batch, was run under the discharge chute of the mixer by a locomotive, whence it was transported to the locomotive cranes which placed the concrete in the forms.

In the construction of the slip walls, the rock on the foundation was thoroughly cleaned of all loose and rotten pieces, washed with a stream of water, and given a coat of mortar. The concrete was brought up in steps of 4 ft. and bonded together by allowing the derrick stone placed in it to project from one course into the other. Steel forms, in sections of 5 by 10 ft., were used, except for the face of the wall for the top 14 ft. Here a wooden form was used, in order to produce a better surface and to facilitate the placing of anchor bolts for the fender system. All the pier and bulkhead walls were built in sections 30 ft. long, each section being tarred to provide an expansion joint.

### Rewatering.

The rewatering of the coffer-dam was effected on June 29th, 1916, by opening the sluice-gates when the tide was at about the low-water stage, which gave a head of 1 ft. of water over the bottom of the sluiceways. The rising of the tide increased the head until, at the expiration of about 6 hours, it had reached about 5 ft., the average during the rise of the tide thus being 3 ft. The clear width of each sluiceway was 7 ft. 8 in. To fill the dam to its capacity of about 8 000 000 cu. ft., 8 hours were required. Practically no movement of the dam was caused by the rewatering.

### Removal of the Coffer-Dam.

Preliminary to pulling the sheet-piling of the dam, the fill in the pockets and in the outshore embankment was removed to a depth of about 10 ft. below low water, in order to relieve some of the pressure, and jet pipes, operated by compressed air, were forced down along the piling nearly their full depth, in order to break the hold of the mud on the piles.

The plant for pulling the sheet-piling comprised a 25-ton and a 40-ton steam lighter for the main portion of the dam, and stiff-leg derricks for the returns extending inshore at the upper and lower ends of the dam, equipped with inverted McKiernan-Terry steam hammers operated by compressed air. The hammers were attached to the piles with 3, 3½, and 4-in. pins, inserted in a single hole burned in each pile about 2 ft. below the top. Various types of grips were also used, but the most successful was a relatively light one furnished by the Lackawanna Steel Company, which was rigged so that it could be opened by pulling a trailing rope when the pile had been pulled out of reach from the ground. This grip, which catches the pile at the top of the web, has been used continuously by one lighter for all but the hardest pulling.

The pulling was started where the piles were from 65 to 70 ft. long, and considerable difficulty was experienced in taking out the first pile, about a week being consumed in trying various piles before one was extracted from the diaphragm of Pocket 28. The pulling of the diaphragms and outshore panels of Pockets 18 to 29 was then accomplished by driving them up with the hammer.

It was found impossible, however, to start any of the piles next to the rip-rap embankment by these means. A 100-ton lighter was

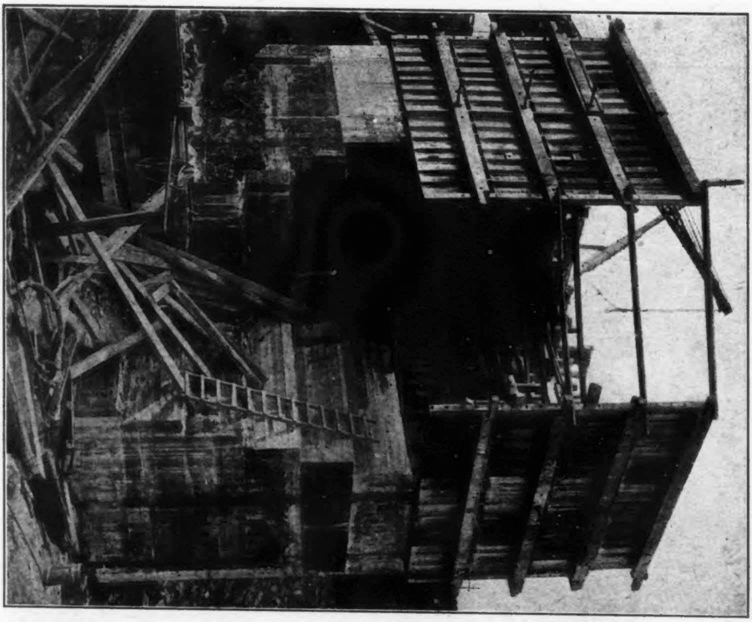


FIG. 23.—OUTSHORE CORNER OF CONCRETE PIER WALL, SEPTEMBER 14TH, 1915.

FIG. 24.—SIDE FACE OF SOUTH PIER WALL, SHOWING COMPLETED CONCRETE, LINE-DRILLED ROCK FACE, AND WALL FORMS ABOVE, NOVEMBER 6TH, 1915.

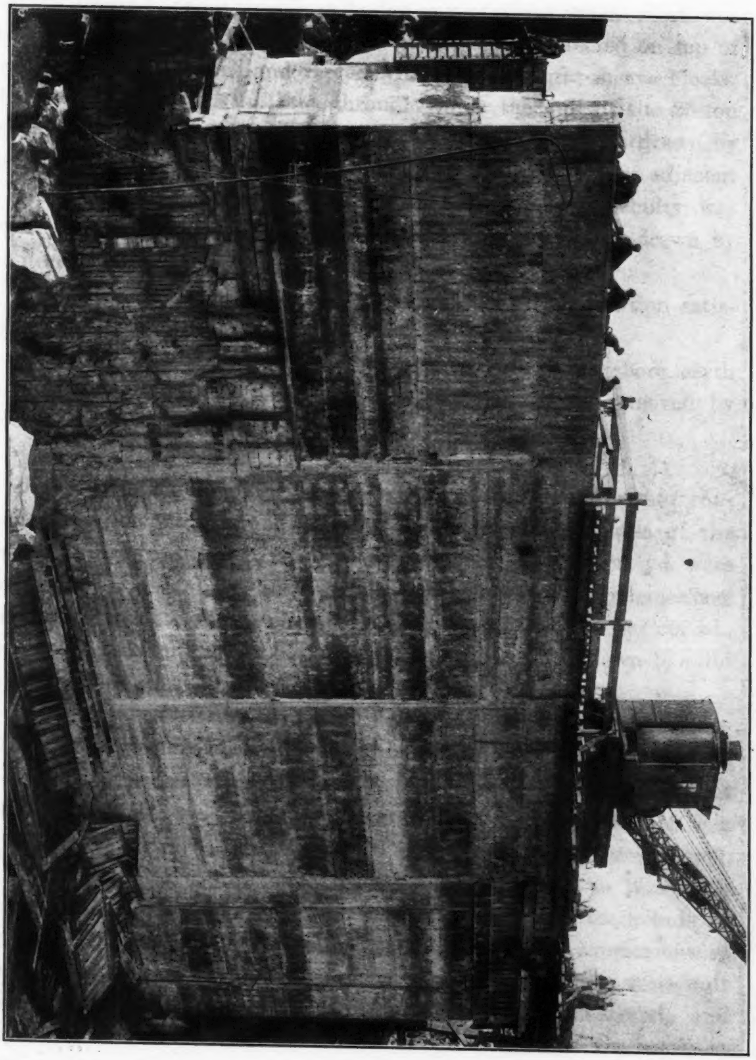


FIG. 25.—ROCK FACE AND CONCRETE RETAINING WALL, NORTH SIDE OF PIER, NOVEMBER 6TH, 1915.

secured with the intention of using it, with an inverted hammer, to break one of the inside arcs. After working 4 days and trying several pockets, this rig, being unable to budge any of these inside piles, was dispensed with. A heavy, guyed, A-frame was then erected on top of the rip-rap embankment and rigged with a set of eight-sheave blocks, with sixteen parts of $\frac{7}{8}$-in. line, through which the pull of the 25-ton derrick was transmitted to the pile. A 64-ft. pile was thus drawn by direct pull without the aid of the hammer. The line of piles adjacent to the rip-rap having thus been broken, no further difficulty was experienced in this direction, and the remaining piles were drawn by the derrick aided by the steam hammer.

As a rule, all the piles came out straight and in a condition satisfactory for use in other work.

After the removal of the sheet-piling, the filling, outshore earth embankment, and inshore rip-rap embankment will be removed by dredging.

## Cost of Work.

*Dredging.*—The dredging was executed under two separate contracts, first for the original contract, and then over the area of the supplementary contract. Under the first contract, 271 882 cu. yd. were removed, at a cost of $23\frac{1}{2}$ cents per cu. yd. Under the supplementary contract, 87 875 cu. yd. were removed, at a cost of 27 cents per cu. yd., making a total, for the removal of material by dredging down to solid rock, of $87 618.52.

*Coffer-dam, Rock Excavation, Walls, etc.*—After due advertisement of the specifications and plans, based on an engineer's estimate of $497 000, for the coffer-dam and walls of the pier, the Department obtained thirteen bids, ranging from $750 000 to $487 000, the low bid, which was awarded to Holbrook, Cabot and Rollins, Incorporated.

In order that the work might be extended southward to West 44th Street, to comprise an area which it was not possible to include at the time of the original preparation of the contract, a supplementary contract was entered into with the same contractors at the same unit prices, after receiving permission from the Board of Estimate and Apportionment and the Board of Aldermen to extend the work in this manner.

It was clearly demonstrated at that time that a saving of fully $150 000 would be made by doing the work in connection with the

original coffer-dam, thereby avoiding the cost of building a new coffer-dam at the south end of the work.

The total of the original and supplementary contracts for the work of building the coffer-dam, excavating the rock, and constructing the concrete walls for the inner part of the pier building, was $626 000.

The unit price for the removal of solid rock under the contract was only $1.60 per cu. yd., but assuming that one-half the cost of the coffer-dam should be charged against rock excavation and the other half against the construction of the concrete walls and other items within the coffer-dam, the actual cost for the removal of this solid rock, was about $5.31 per cu. yd.

## Conclusion.

*First.*—Instead of excavating this large quantity of subaqueous rock by the "dry" method, the work might have been performed without a coffer-dam by the ordinary method of rock blasting under water, or, in other words, the excavation of the rock "wet".

If it were possible to perform this work so as to obtain under water vertical face walls to the exact limits of the slip, it would have cost, under ordinary normal prices for this class of work, four or five times as much as by the method adopted.

It is not possible to conceive, however, that the work could have been done under water, for, as a matter of fact, it was very difficult to do it with line drilling in the open, and further, even were it possible to deposit concrete in a satisfactory manner at such depths, part of the work in addition would have been very expensive.

The particular result obtained by the construction of this dam was that, although all preliminary examinations, borings, soundings, etc., were made when the whole territory was covered by filling of all kinds, including, in places, cribwork, coal yards covered with coal, buildings, etc., the total amount paid under the contract obtained through unit prices has been kept close to the original estimates.

*Second.*—The whole river face of this dam, for a length of about 800 ft., under its full water head, was absolutely dry, all the pumping, during the whole construction period having been done by one 12-in. centrifugal pump. Furthermore, all this pumping would have been avoided, although much more was expected, had it not been for the presence of cribwork at the north side, and on account of the impossi-

bility of driving two piles in the large cylinders at the south end, because, in driving, a boulder or some other interference was encountered, rendering it impossible to drive these two piles home for a distance of only about 1 ft.

## Supervision.

The work was executed by the Department of Docks and Ferries, R. A. C. Smith, Commissioner of Docks, and R. C. Harrison, First Deputy Commissioner.

The design, preparation of plans and specifications, and the construction was under the direction of the writer, as Chief Engineer, assisted by R. T. Betts, M. Am. Soc. C. E., Deputy Chief Engineer; Mr. T. F. Keller and Elias Cahn, Assoc. M. Am. Soc. C. E., Assistant Engineers. The work in the field was under the direction of J. J. Pemoff, M. Am. Soc. C. E., Assistant Engineer.

The contracting firm for the work was Holbrook, Cabot and Rollins, Incorporated.

Acknowledgment is made by the writer of the skill and perseverance of the contractors, particularly of T. B. Bryson, Assoc. M. Am. Soc. C. E., on the work. The writer's thanks are also due to Brig.-Gen. William M. Black, M. Am. Soc. C. E., Chief of Engineers, U. S. A., for advice in the preparation of the plans, and the support of his assurance that this work could be accomplished successfully provided proper construction methods were used.

bility of driving two piles in the large cylinders at the south end, because in driving, a boulder or some other interference was encountered, rendering it impossible to drive these two piles home, the [illegible] dimension being [illegible] it.

ACKNOWLEDGMENTS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# THE RECONSTRUCTION OF THE STONY RIVER DAM

By F. W. Scheidenhelm, M. Am. Soc. C. E.

To be Presented March 21st, 1917.

## Synopsis.

This paper describes the reconstruction and strengthening of the Stony River Dam, which failed on January 15th, 1914. After the failure, on investigation of the foundation conditions and the parts of the structure which remained intact, it was found that those portions required strengthening and revision in a number of important features. The lessons indicated by the results of the investigation are significant. The dam, even as reconstructed and strengthened, is founded largely on clayey soil, thus presenting most difficult foundation conditions. It is about 50 ft. high above stream level and 85 ft. between the lowest part of the cut-off wall and the top of the parapet.

A brief résumé of the previous history of the dam, including the causes of its failure, is first given, together with a statement of the controlling geological and foundation conditions. Then follows an exposition of the treatment of the more important problems involved in the reconstruction.

The spillway capacity was increased approximately to 1 840 cu. ft. per sec. per sq. mile. The reasons for providing such unusually large capacity are given, and also a method of determining the absorption effect of a reservoir in smoothing off the peak of a flood.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The most important problem to be solved was that of giving the original structure a sufficient margin of safety against sliding. The results of tests of frictional resistance and shearing value of various soils, principally clays, are stated, and also the various schemes considered for increasing the resistance to sliding. The method adopted is believed to be new, and consists in the use of anchoring walls extending to a considerable depth into the underlying foundation soil, and, in effect, utilizing the weight of that underlying soil as well as the resistance (to horizontal movement) of the soil immediately down stream from the structure. The details of construction are unique. The same principles were applied, with important differences in detail, at the new spillway which replaced that portion of the structure which had failed or was wrecked.

Brief reference is made to the resistance of the structure—both in its original state and as strengthened—to failure by overturning.

The bearing value of clayey soil is discussed, and the results of tests made at the site are given. The anchoring wall added at the toe of the original structure was utilized to decrease the pressures of the foundation soil. The method of strengthening the original footings is described. Pressure grouting was used to remedy other faulty footing conditions.

The means of cutting off percolation and leakage through the foundation soil are described, also the provision made for drainage, both of clayey soil and of shale rock, together with the controlling considerations in these and similar cases.

The higher bulkhead portions of the original structure were housed in by curtain-walls and roofs in order to prevent serious freezing in the drainage system and to fulfill certain other functions as described.

Miscellaneous problems in the reconstruction and strengthening involved the type of structure for the new spillway, the strength of the original decks and buttresses, the underpinning of the portions of the original structure immediately adjoining the new spillway, the construction of a protecting and anchoring toe-wall at the old spillway, the special method of constructing an anchoring wall at the heel, at the outlet-gate sections, and the provision of an outlet channel.

The reliable storage capacity of the reservoir was increased by 25 per cent. This increase was made permissible by the use of special steel bars or pins acting as flash-board supports. These pins were

developed for the purpose, and automatically and reliably allow the flash-boards to be swept off the spillway crests when the head-water reaches a pre-determined level.

The materials, methods, and cost of reconstruction are discussed briefly. Approximately, as much concrete was required for the reconstruction as had been placed in the original structure.

The Stony River Dam and Reservoir have again been in service since May, 1915, and the observed results of the reconstruction and strengthening are stated in connection with the corresponding features of design.

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## Reconstruction of Stony River Dam.

The failure and reconstruction of the Stony River Dam are of interest, not because of the height or dimensions of the structure, but because it is largely through failures and mistakes that engineers must learn the limitations of previous methods of design and construction and gather hints as to new methods to be used. Furthermore, in the case of this structure, the extremely difficult foundation conditions made the problem of providing a safe dam at that site one requiring more than usual thought and care. Finally, much interest attaches to the methods used to make safe the existing structure, that is, that portion which remained intact after the failure—it being noted that a comparatively small section of the dam actually failed.

It seems proper at this point to emphasize the fact that, until a structure has actually failed, its "safety" is relative. Seldom do any two engineers have exactly the same conception or measure as to the factor or margin of safety possessed by a given structure. Especially is this true in the case of dams.

As a preface to the description of the reconstruction and the problems involved therein, the following brief review of the previous history of the dam is pertinent. The details have been gathered from various sources, and are based on the best information available to the writer.

### Previous History.

In 1911 the West Virginia Pulp and Paper Company, owner of the dam, determined to investigate the feasibility of constructing on the head-waters of the North Branch of the Potomac River a storage reservoir by which to increase the flow of the river at its Piedmont

pulp and paper mill at Luke, Md., near Piedmont, W. Va. Its requirements of water for manufacturing purposes were such that during low-water seasons the normal flow of the stream was at times inadequate.

After an investigation of the available sites on the drainage area above the Piedmont mill, the owner chose the water-shed of Stony River as offering the best conditions for the construction of a storage reservoir. Stony River—which at the site of the dam drains only 11.4 sq. miles, and is therefore only a creek—lies entirely within Grant County, West Virginia, flowing into the North Branch of the Potomac River from the south a short distance above Schell, W. Va. The mouth of Stony River is approximately 26 miles up stream from the Piedmont mill. It is only near its upper end that the profile is flat enough to allow the construction of a reservoir at a practicable cost per unit of water stored.

Two sites examined in the upper reaches of Stony River were abandoned, because of interference with a seam of commercial coal in one case and the presence of quicksand in the over-burden in the other. The third and final site was chosen with the approval of Edward Wegmann, M. Am. Soc. C. E., who had been engaged by the owner as Consulting Engineer for the purpose.* This site is approximately 3 350 ft. above sea level.

An examination of the foundation conditions, deemed at the time to be adequate, was made by test pits and auger borings. After this examination it was concluded that the depth to bed-rock was so great as to prohibit the construction of a solid masonry dam, and that the surface soil in the neighborhood of the dam was not of such a nature as to warrant the construction of an earth fill dam. It had been the intention of the Company to build a dam of one of these types, but, under the circumstances, it was decided to construct a hollow reinforced concrete structure with a height of about 50 ft. above the original water surface and a length of about 1 075 ft. The resulting capacity was approximately 1 533 000 000 gal. below the elevation of the spillway crest.

Certain companies were invited to submit plans and bids for the construction of such a dam. On the basis of the bids received,† a contract was entered into with Mr. Frederick G. Webber, President of the

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1069.

† *Engineering News*, September 5th, 1912.

Webber Construction Company, for the construction of a dam according to plans submitted by the Ambursen Hydraulic Construction Company, licensor.

Work was begun by the contractor about June 1st, 1912, and carried on until February, 1913. At that time the Company took the work from the original contractor and engaged the Ambursen Hydraulic Construction Company to complete the work. The latter company took over the work in March, 1913, and completed it in July, 1913.

The details of the original construction have been fully described by G. H. Bayles, M. Am. Soc. C. E., Resident Engineer for the owner during the construction.* Variations of actual conditions from those assumed in the original design and construction will be discussed later in this paper. Fig. 1 shows the dam as originally built.

The storage of water in the reservoir was begun in May, 1913, that is, prior to the completion of the dam. The reservoir, however, was not completely filled until the late autumn, and the water level had been at the spillway crest for about 65 days prior to the failure of the dam. There had been but little flow over the spillway.

The failure occurred on January 15th, 1914, the cause, in the opinion of the writer, being the washing out of the foundation soil by leakage from the reservoir under an up-stream cut-off wall of inadequate depth. It is probable that there were conclusive signs of impending trouble for several days earlier (perhaps as early as January 10th), in the way of water flowing through weep-holes in the floor or footing of the dam at or near the section where failure finally occurred. With proper attention on the part of the operating attendant at the dam, the reservoir could probably have been drained in time to avoid the failure which occurred.

It is not the purpose in this paper to discuss the failure in detail, inasmuch as it has been described adequately in articles in engineering periodicals.† For the sake, however, of making clear the description of the design, and the work involved in the reconstruction, Figs. 2, 3, and 4 are submitted, showing the portion of the dam which failed.

As a matter of personal interest, the writer visited the site as soon as practicable after the failure, and spent portions of January 18th and 19th, 1914, there. At the time of his inspection the main outlet-

* *Engineering News*, January 22d, 1914.

† *Engineering News*, January 22d, 1914; *Engineering Record*, January 24th, 1914.

gate in the dam had been opened, and the reservoir had already been completely drained, so that Stony River was flowing in its normal channel at the center of the valley, and not through the break, which occurred at a higher level, and on the west bank of the valley. Based on the inspection made at that time, the writer prepared and furnished to the Public Service Commission of West Virginia, at its request, his conclusions as to the failure of the dam. These conclusions, dated January 22d, 1914, were as follows:

1.—Complete failure occurred between Buttresses 11 and 16, involving a length of approximately 75 ft. out of a total crest length of approximately 1 075 ft. At least three additional bays, *viz.*, as far eastward as Buttress 19, were damaged so as to require practically complete rebuilding. Similarly, the faulty cut-off wall construction and certain damage done westwardly from Buttress 11 must be remedied.

2.—Failure was caused by the undermining of the over-burden or soil under the up-stream cut-off wall. The over-burden is in general clayey, but non-homogeneous. Undermining was initiated by leakage of water from the reservoir under approximately 25 ft. head through permeable over-burden not penetrated by the cut-off wall.

3.—The pressure of ice on the reservoir against the deck of the dam did not contribute to the failure.

4.—The dam was of the Ambursen, hollow, reinforced concrete type. The type of dam was not in any way a cause of failure.

5.—The quality of concrete and character of construction were good.

6.—Failure occurred where the up-stream cut-off wall extended only a short depth (5 to 7 ft.) into the over-burden. Where the up-stream cut-off wall extended to a comparatively greater depth (said to be to rock), failure was checked, and the cut-off wall is still intact.

7.—The preliminary exploration and investigation of the dam site were not sufficiently comprehensive, and did not develop essential and all-important facts.

8.—The technical advisers consulted in connection with the foundation of the dam were not familiar with local conditions. The

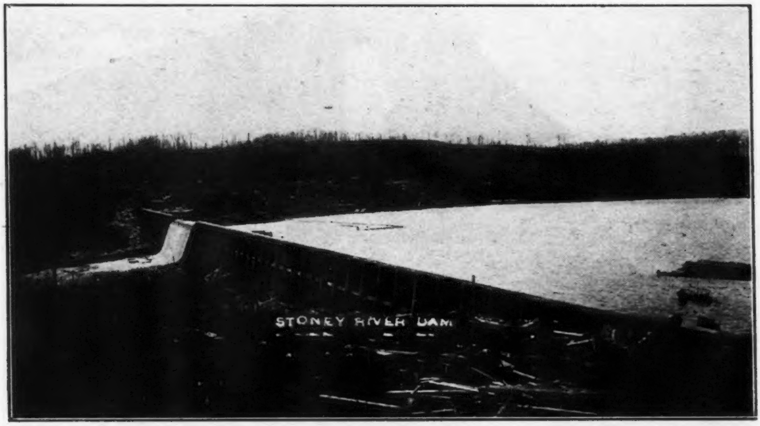



FIG. 1.—STONY RIVER DAM, AS ORIGINALLY CONSTRUCTED.

FIG. 2.—BREAK AND WASH-OUT OF STONY RIVER DAM, JANUARY 18TH, 1914.

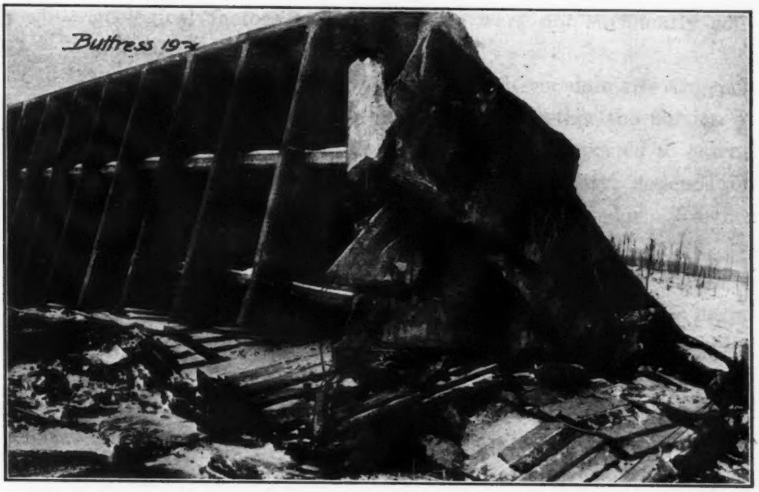



FIG. 3.—EASTERN EDGE OF BREAK IN STONY RIVER DAM, JANUARY 18TH, 1914.

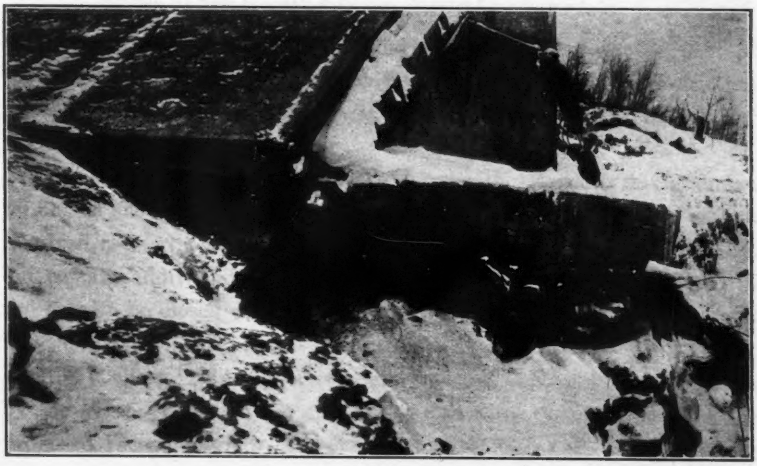


FIG. 4.—BUTTRESS 11 OF STONY RIVER DAM, JANUARY 18TH, 1914.

geological factors at the dam site were not sufficiently considered.

9.—The conditions obtaining at the Stony River dam site are such that a safe dam could be built there. Whether the portion of the dam still intact, and with its core-wall reported to extend down to rock, has sufficient factors of safety, depends on facts of which we do not have accurate knowledge. Further investigation should be made as to this question.

10.—The mechanism for operating the outlet-gates and valves is not sufficiently protected against ice formed by otherwise harmless leakage through the deck of the dam; nor is such mechanism readily accessible under all conditions.

11.—Although the dam should, and could, have been designed and constructed so as to be absolutely safe, regardless of the gate mechanism and the character of the attendance, yet, even in this instance, had the attendant been observant and resourceful, it would have been possible to open the sluice-gates in ample time to drain the reservoir sufficiently to have obviated failure of the dam.

12.—Published newspaper reports have grossly exaggerated the details of the failure and the extent of the damage to property.

In general, the foregoing summarized conclusions on the part of the writer still hold good. It may be well to point out, however, that the writer distinguishes between the "character of construction work" and the design of the structure.

The location of the break in the dam, with reference to the entire profile, is shown on Plate II. Complete failure, as previously stated, involved a length of about 75 ft., *viz.*, between Buttresses 11 and 16, but certain additional damaged work was removed, so that the gap in the dam to be closed during the reconstruction work comprised a total length of 135 ft., *viz.*, between Buttresses 10 and 19.

In February, 1914, the writer was engaged as Consulting Engineer by the West Virginia Pulp and Paper Company to take charge of the proposed reconstruction. He desires to emphasize the point that he was not requested to place the responsibility for the failure, nor did he attempt to do so. It is fair to note, however, that in this case, unlike most other failures, no criticism has been heard directed against the

owner, that is, against the executive officers of the West Virginia Pulp and Paper Company, on the score that economy was preferred to safety. Moreover, in connection with the proposed reconstruction, the executive officers of the Company proceeded on the principle that either the dam should be reconstructed so as to be reliably safe, or no attempt should be made to utilize the damaged structure. The same attitude was manifested throughout the entire period of reconstruction.

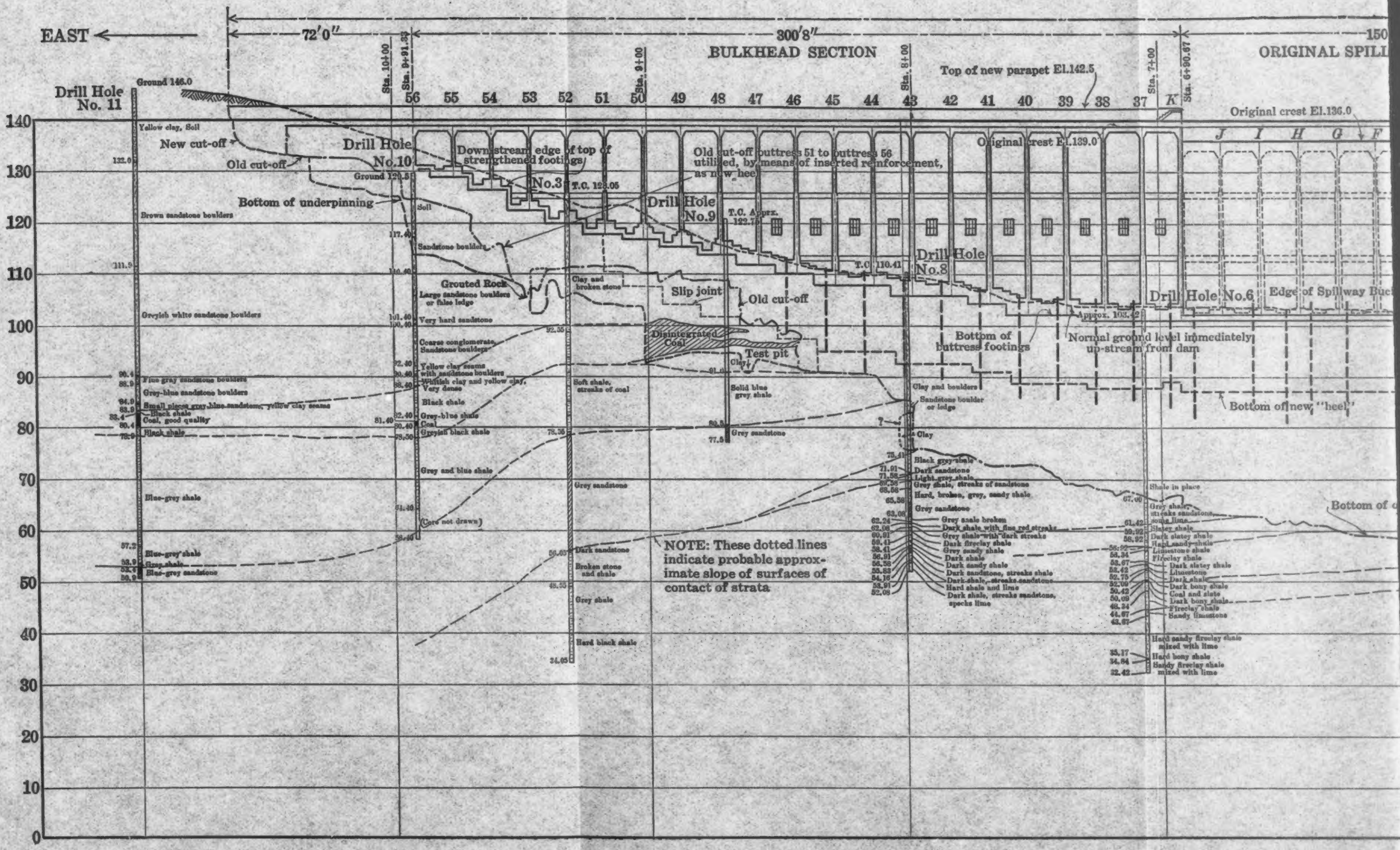
Prior to the final decision to proceed with the actual work of reconstruction, the entire structure and foundation conditions were investigated carefully, in order to discover any features of design or construction involving insufficient margins of safety. Not until about August 1st, 1914, did the owner finally decide to reconstruct. The work was finished in May, 1915, though the storage of water in the reservoir had been resumed several months earlier.

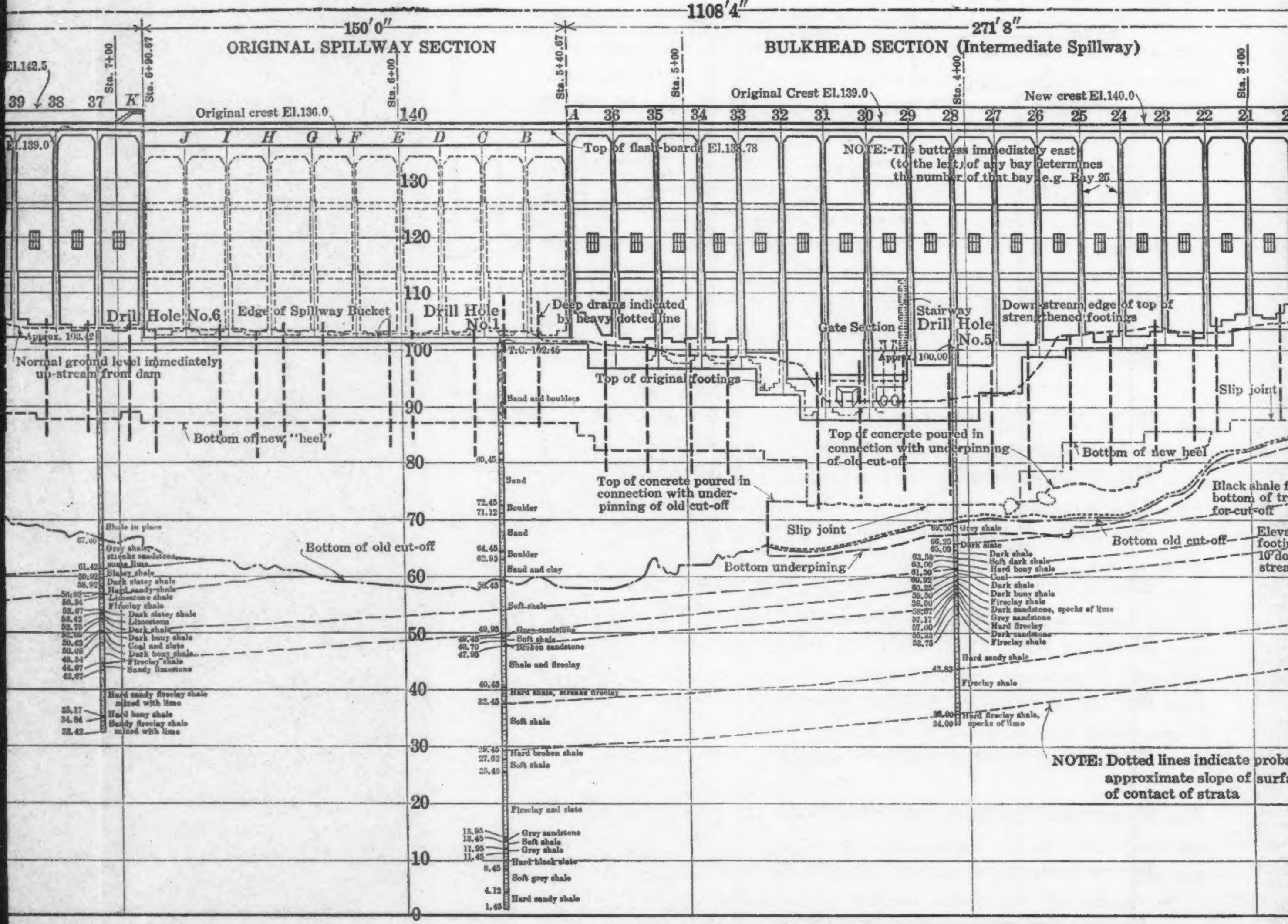
## Geological and Foundation Conditions.

The available information regarding foundation conditions was found to be so inadequate that the investigation which followed had to be made practically as complete as though there had been no dam on the site. In fact, the existence of a dam at the site necessitated additional exploration to determine the adequacy of the cut-off.

An F-1 Davis Calyx core-drill was placed on the work, and by it ten holes were sunk. The deepest hole was sunk to approximately 100 ft. below ground surface; several other holes were carried to depths of approximately 80 ft. Usually, the strata penetrated were sufficiently tight to retain in the hole the water necessary for the shot-drilling process. In other cases, however, especially on the east bank of the valley, seams were encountered in the rock, and these were of such capacity that it was difficult to keep sufficient water in the holes to allow the drilling to proceed.

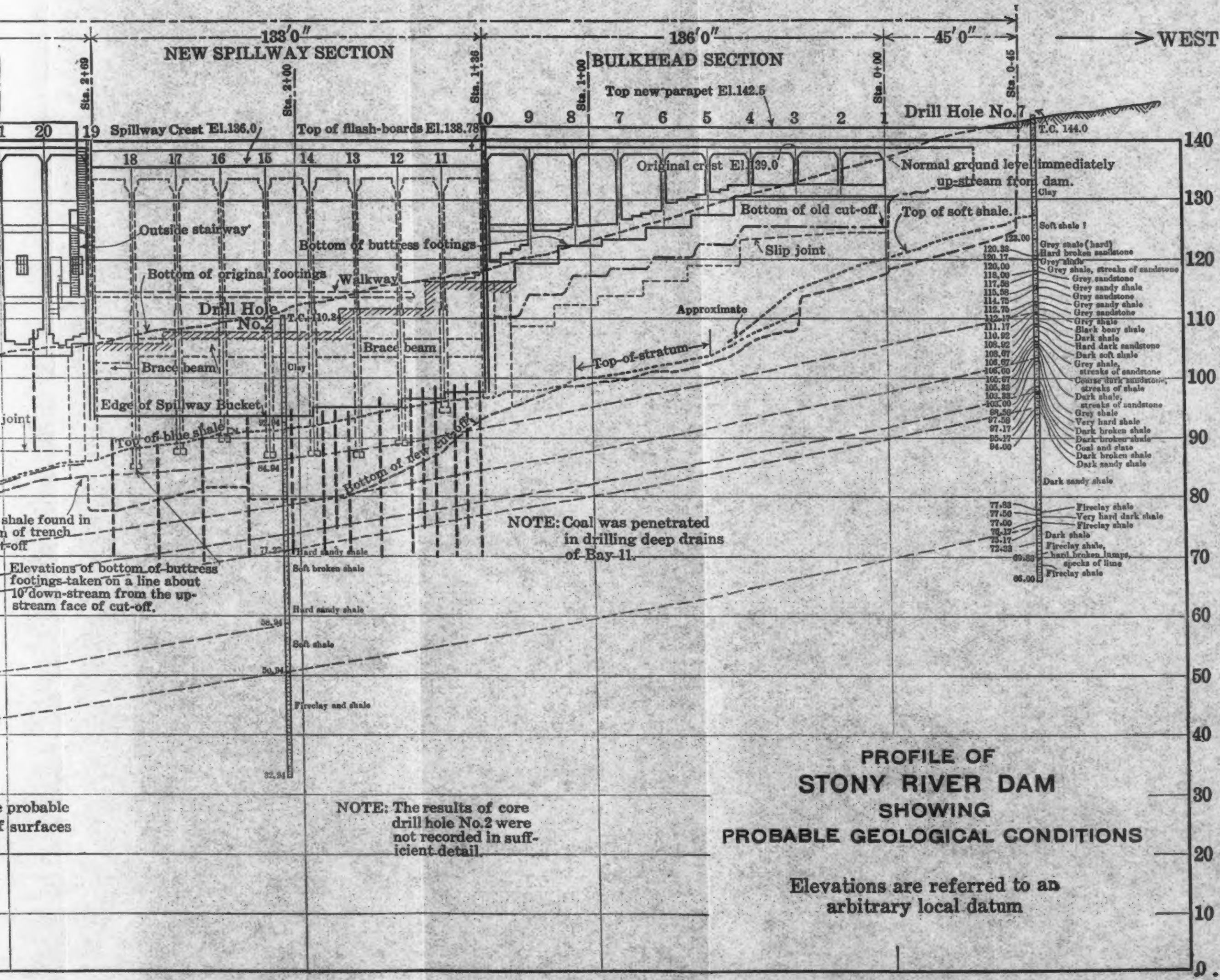
Numerous test pits were sunk, the majority being immediately adjacent to, and up stream from, the original cut-off wall. Such test pits not only exposed the character of the over-burden, but also permitted an examination of the original cut-off and the efficacy of its seal into the bed-rock. Several of the test pits extended to a depth of approximately 45 ft., and were sunk at considerable expense, because of the inflow of water and the necessity for sheeting the pits for practically their entire depth.


EAST
72'0"
300'8"
150
BULKHEAD SECTION
ORIGINAL SPILL
Top of new parapet El.142.5
Original crest El.136.0
Drill Hole No. 11
Ground 146.0
New cut-off
Old cut-off
Drill Hole No.10
Downstream edge of top of strengthened footings
No.3
Old cut-off buttress 51 to buttress 56 utilized, by means of inserted reinforcement, as new heel
Original crest El.139.0
Bottom of underpinning
Drill Hole No.9
Drill Hole No.8
Grouted Rock
Slip joint
Old cut-off
Drill Hole No.6
Edge of Spillway Bucket
Disintegrated Coal
Test pit
Bottom of buttress footings
Normal ground level immediately up-stream from dam
Approx. 103.42
Bottom of new "heel"
NOTE: These dotted lines indicate probable approximate slope of surfaces of contact of strata
Yellow clay, Soil
Brown sandstone boulders
Greyish white sandstone boulders
Blue-grey shale
Grey and blue shale
Grey sandstone
Dark sandstone
Grey shale
Hard black shale
Solid blue grey shale
Grey sandstone
Clay and boulders
Shale in place



1108′4″
150′0″
ORIGINAL SPILLWAY SECTION
271′8″
BULKHEAD SECTION (Intermediate Spillway)
El.142.5
El.139.0
Sta. 7+00
Sta. 6+90.67
Sta. 6+00
Sta. 5+40.67
Sta. 5+00
Sta. 4+00
Sta. 3+00
Original crest El.136.0
Original Crest El.139.0
New crest El.140.0
Top of flash-boards El.138.78
NOTE:-The buttress immediately east (to the left) of any bay determines the number of that bay, e.g. Bay 25
Drill Hole No.6
Edge of Spillway Bucket
Drill Hole No.1
Deep drains indicated by heavy dotted line
Gate Section
Stairway
Drill Hole No.5
Down-stream edge of top of strengthened footings
Approx. 103.42
Approx. 100.00
T.C.-102.45
Normal ground level immediately up-stream from dam
Top of original footings
Slip joint
Bottom of new "heel"
Top of concrete poured in connection with underpinning of old cut-off
Bottom of new heel
Top of concrete poured in connection with under-pinning of old cut-off
Slip joint
Bottom of old cut-off
Bottom underpining
Bottom old cut-off
Sand and boulders
Sand
Boulder
Sand
Boulder
Sand and clay
Soft shale
Grey sandstone
Soft shale
Broken sandstone
Shale and fireclay
Hard shale, streaks fireclay
Soft shale
Hard broken shale
Soft shale
Fireclay and slate
Grey sandstone
Soft shale
Grey shale
Hard black slate
Soft grey shale
Hard sandy shale
Shale in place
Grey shale, streaks sandstone, some lime
Slatey shale
Dark slatey shale
Hard sandy shale
Limestone shale
Fireclay shale
Dark slatey shale
Limestone
Dark shale
Dark bony shale
Coal and slate
Dark bony shale
Fireclay shale
Sandy limestone
Hard sandy fireclay shale mixed with lime
Hard bony shale
Sandy fireclay shale mixed with lime
Grey shale
Dark slate
Dark shale
Soft dark shale
Hard bony shale
Coal
Dark shale
Dark bony shale
Fireclay shale
Dark sandstone, specks of lime
Grey sandstone
Hard fireclay
Dark sandstone
Fireclay shale
Hard sandy shale
Fireclay shale
Hard fireclay shale, specks of lime
NOTE: Dotted lines indicate proba approximate slope of surfa of contact of strata

PLATE II.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
SCHEIDENHELM ON
RECONSTRUCTION OF
STONY RIVER DAM.




PROFILE OF
STONY RIVER DAM
SHOWING
PROBABLE GEOLOGICAL CONDITIONS

Elevations are referred to an arbitrary local datum

On Plate II are shown the locations of the core-drill holes, the logs thereof, and the "lay" of the rock strata as deduced from the logs and other records. This plate shows, also, the profile of the original ground surface of the valley. It is apparent that the axes of the original valley and of the present valley do not coincide. By "original" valley is meant that valley which existed at the time the stream was still eroding the bed-rock and was in immediate contact with that rock. At that time the axis of the valley was approximately at the west end of the old spillway, or at about Station 5 + 50. As time went on, the stream ceased its cutting or eroding action and began to build up the floor of the valley by depositing sediment and gravel. At the sides of the valley this process was apparently aided materially by "slips" and talus from the hillsides.

During this process Stony River meandered back and forth across the valley until at present the axis of the valley is approximately at Station 4 + 40. If, in the profile shown on Plate II, the original and the present locations of the stream-bed were to be connected by a line passing through all the intermediate locations of the stream-bed, such connecting line would have an extremely irregular or zigzag course. Water-worn boulders and gravel were found strewn through various portions of the over-burden as exposed by the exploratory work, testifying to the fact that they had been deposited by the stream in ages past.

The Stony River Valley lies in the geological formations of the Carboniferous period. The lower portion of the valley, at the dam site, lies in what is known locally as the Savage formation, one of the principal features of which is the Davis or Kittanning coal, which is of commercial importance throughout this region.

The general characteristics and slope of the formations are indicated roughly in the Piedmont folio (West Virginia) of the United States Geological Survey. The detailed geological correlations at the dam site, however, were far from simple. Variability, rather than uniformity or persistency, characterizes the Savage formation, and nowhere is this exemplified better than at the site in question. From the core-drilling data it is apparent that lentils abound, that relatively soft strata are interbedded with hard strata, and, what is of even greater importance, that the geological horizons, which at one location offer impervious strata, show decidedly pervious strata at other locations.

The characteristic rocks are shales of a laminated or seamy character, as shown in the typical section, Fig. 6. Here and there lentils of coal and black slate are interbedded in them. At approximately the location of the break, *viz.*, in the neighborhood of Station 2 + 00, the shales were found to be so crumpled and crushed by folding near the surface as to require their excavation in the process of reconstruction.

It is not believed that this condition had any relation to the failure of the dam, but it is probably true that, once the dam had failed and the reservoir was being emptied through the break, the erosion of the bed-rock immediately down stream from the place of failure was facilitated considerably by this local surface crumpling of the shales.

At the east side of the valley (the left side of the profile in Plate II) heavy sandstones were encountered. Evidently, at the time the valley was formed by the process of stream erosion, the sandstones caused the formation of overhanging cliffs, under which the softer strata were eroded. Frost and other natural forces, however, had their effect, even on these sandstone cliffs, with the result that they became fissured, though the boulders which were thus cracked off probably did not move far from their original location. In fact, excavations made under the original cut-off showed that in the original construction certain of these boulders or false cliffs had been mistaken for bed-rock, and that under them there was actually pervious material, such as the disintegrated coal shown in the profile immediately to the west of Station 9 + 00.

The over-burden or foundation soil at this site is mainly of a clayey nature. It is far from being homogeneous, as is evident from Fig. 7, which shows typical soil conditions at Buttress 11. In general, the soil near the surface is loamy and more pervious than that immediately overlying the bed-rock. Near the middle of the valley, however, the conditions are reversed, and there deposits of pervious materials, such as gravels, were found at considerable depths.

A catalogue of the constituent materials of the over-burden shows:

Yellow clay...........impervious, of fair bearing value.

Blue clay.............impervious, of fair bearing value.

Black clay or gumbo..impervious, of fair bearing value when dry or only moist, but of poor bearing value when saturated with water.

Sand and gravel......pervious, but of good bearing value.

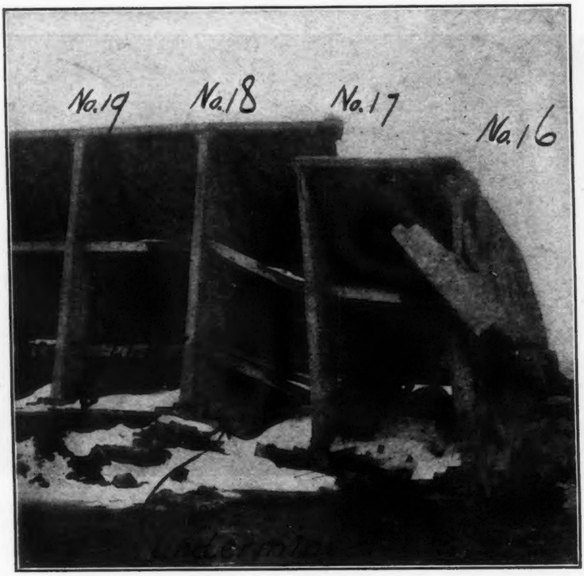



FIG. 5.—BUTTRESSES 16 TO 19, STONY RIVER DAM, MARCH 16TH, 1914.

FIG. 6.—TYPICAL SECTION OF SHALE AT SITE OF STONY RIVER DAM, NEAR POINT OF FAILURE.

FIG. 7.—TYPICAL FOUNDATION SOIL CONDITIONS AT BUTTRESS 11, STONY RIVER DAM.

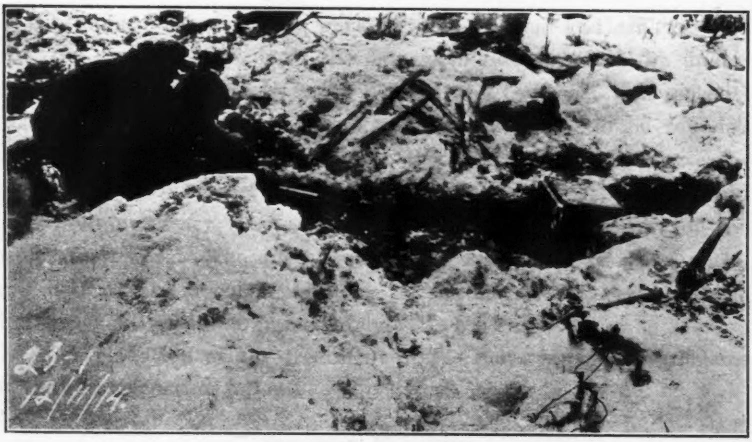



FIG. 8.—TEST OF FRICTIONAL RESISTANCE.

Loamy sand.......... in minor quantities, pervious, of poor bearing value.

Sandy clay........... fairly water-tight, of fair bearing value.

Shale fragments....... not in place, and in various stages of disintegration, mostly permeable, of fair bearing value.

Boulders............. in considerable numbers, and, of course, of excellent bearing value.

The bearing values, as referred to in the foregoing, are relative; comparison is made with the bearing values of other materials present in the over-burden, and not with those of granite or even of the shale rock on which the new buttresses, 11 to 18, inclusive, are founded. The materials occurred in no regular order.

Such foundation conditions required the most serious consideration and care in construction. In general, the designs involved in the work of reconstruction were based on the assumption that the worst conditions might exist at any bay. In view of the findings of the exploration work, it appeared logical, however, to attribute higher bearing values and greater resistance to percolation to the foundation soil of the east hillside, that is, roughly, east of Station 5 + 00, for in that portion of the valley there is more clay, and it is of a better character.

Simultaneously with the work of core-drilling and sinking test pits, the wrecked portions of the dam were removed, and the underlying foundation soil was stripped from the bed-rock, thus permitting a first-hand study of the strata on which were to be founded the buttresses which were to replace those taken out. Such excavation was actually a part of the reconstruction, inasmuch as it would have been necessary later, if it had not been made as part of the exploratory work. The investigation was completed in June, 1914, and designs and recommendations for reconstruction based on the findings were prepared immediately.

An explanation of the treatment of the more important problems of the reconstruction follows.

### Spillway Provision.

*Original Spillway Capacity.*—A problem affecting most of the other features of design—because it determined the probable maxi-

mum head-water level, and hence affected the probable maximum stresses—was that of the spillway capacity to be provided. The original spillway provision was apparently inadequate, being only about 2 800 sec-ft. with water level at the elevation of the original crest of the bulkhead sections of the dam, *viz.*, Elevation 139 (Plate II). In fact, it is likely that the actual safe flood discharging capacity would have been somewhat less, for the original spillway crest had not been given the most advantageous shape.

Assuming the drainage area to be 11.4 sq. miles, as shown by the "Piedmont" topographical quadrangle issued by the U. S. Geological Survey, the original spillway capacity was less than 250 sec-ft. per sq. mile of drainage area.

*Pertinent Records of Flood Flow.*—In general, in a given region, the smaller the drainage area the larger the maximum unit run-off and flood flow. So-called cloudbursts are limited as to area affected, but certainly the Stony River drainage area is sufficiently small to fall within the territorial limits of a cloudburst. It happens that there are two important and applicable records* of flood discharge, namely, those of the floods on:

Cane Creek, at Bakersville, N. C., May 19th-20th, 1901; drainage area, 22 sq. miles; estimated maximum discharge, 1 386 sec-ft. per sq. mile; elevation above sea level at point of measurement, approximately 2 450 ft. (U. S. G. S. datum); elevation of highest point in water-shed, approximately 5 330 ft.; and

Elkhorn Creek, at Keystone, W. Va., June 22d, 1901; drainage area, 44 sq. miles; estimated maximum discharge, 1 363 sec-ft. per sq. mile; elevation at point of measurement, approximately 1 600 ft.; elevation of highest point in water-shed, approximately 3 365 ft.

The fact that both of these extreme floods occurred in the Southern Appalachian mountain system, in which the Stony River dam is situated, and the further fact that the several drainage areas in question, including that of Stony River (with the dam at approximately Elevation 3 350 and the highest point in the water-shed at approximately Elevation 4 200), are at not widely differing altitudes, cause

* *Engineering News*, August 7th, 1902.

these two records to be exceptionally valuable for purposes of comparison. These drainage areas are apparently quite similar in their physical characteristics.

The records referred to were reported by Mr. E. W. Myers, then Engineer of the North Carolina Geological Survey, and Assistant Hydrographer of the U. S. Geological Survey. It is to be admitted that the method of measurement was probably not such as to warrant expressing the discharges to the nearest second-foot; on the other hand, an examination of the data does not warrant one in disregarding the records or assuming that the maximum flood discharges were materially less than those stated.

Although the Cane Creek flood was estimated to have been actually of slightly greater intensity than that of Elkhorn Creek, yet relatively, and for the purpose of predicting the maximum probable flood discharge from the drainage area above the Stony River dam, the Elkhorn Creek flood is of considerably greater importance, because of the fact that the drainage area at the point of measurement, is twice as great as in the case of Cane Creek. For detailed data, however, one must work from the records of the Cane Creek flood, as in that case, fortunately, Mr. Myers made an estimate of the average discharge during each hour of the flood, as well as an estimate of the maximum discharge during the flood. In the graph of this flood, Fig. 12, the run-off of Cane Creek has been reduced in proportion to the drainage area, so as to be applicable to the drainage area above the Stony River Dam. Granting that the Cane Creek data are only approximate, they nevertheless appear to be well worth while adopting as a basis of study.

It is noteworthy that all available records indicate that in the region under consideration maximum rains, of sufficient duration to tax the equalizing capacity of the reservoir, do not occur in the early spring, and hence do not coincide with the melting of the snow. For this reason it was assumed that the greatest flood to be anticipated at the Stony River Dam would be due solely to rainfall.

*Comparison with Records of Maximum Rainfall.*—For practical purposes, it may be assumed that the Elkhorn and Cane Creek floods were of equal intensity when reduced to a unit basis. The following remarks, based on the Cane Creek record, are consequently applicable also to the even more important Elkhorn Creek record.

Both these floods were undoubtedly caused by local cloudbursts which occurred after previous rains had saturated the respective drainage areas. For the purpose of comparison with rainfall records, therefore, it may be assumed that the maximum run-off per hour was approximately equal to the maximum rainfall per hour. In the case of the Cane Creek flood it is reported that, during the hour of maximum run-off, the discharge was at an average rate of 1 202 sec-ft. per sq. mile, corresponding to a rainfall at the rate of approximately 1.86 in. per hour.

Referring to Fig. 9, showing the duration of maximum intensities of rainfall in the United States, it will be noted that the maximum rate of rainfall previously derived for the Cane Creek flood falls well within the limits of the highest records shown.

*Bulletin D* of the U. S. Weather Bureau, the basis of the principal data of Fig. 9, includes only records obtained after the establishment of self-registering gauges in 1893, but prior to 1897. Because of the limited period which it covers, and, further, because of the limited number of precipitation stations involved, it is certain that the data of *Bulletin D* are only indicative, and not comprehensive or conclusive.

This fact is illustrated further by the record of an excessive rainfall at Raleigh, N. C., on Tuesday, July 14th, 1914, which has been plotted on Fig. 9. It should be remembered, of course, that records of excessive rainfall such as that at Raleigh are not of controlling importance in the present case, because they are not of sufficient duration to tax the equalizing capacity of the reservoir.

On Fig. 9 there has also been plotted a curve (*b*), derived from the graph shown on Fig. 12, on the assumptions that during the 90 min. of greatest run-off, in the case of the Cane Creek flood, the rainfall likewise was at its maximum, and that during such period the run-off was equal to the rainfall, variations being simultaneous. These assumptions, of course, are not strictly true, as the maximum rate of run-off could not have been equal to the maximum rate of rainfall. Necessarily, there must be some equalization, and it is evident from an inspection of the Cane Creek flood curve (*b*) of Fig. 9 that in this instance there must have been a considerable difference between the maximum rates of run-off and rainfall.

Yet, even allowing for a reasonable equalization or smoothing off of the peak of the rainfall, as compared with the peak of the run-off, it is apparent that the rainfall which caused the Cane Creek flood was by no means as great as may be expected in either the Cane Creek

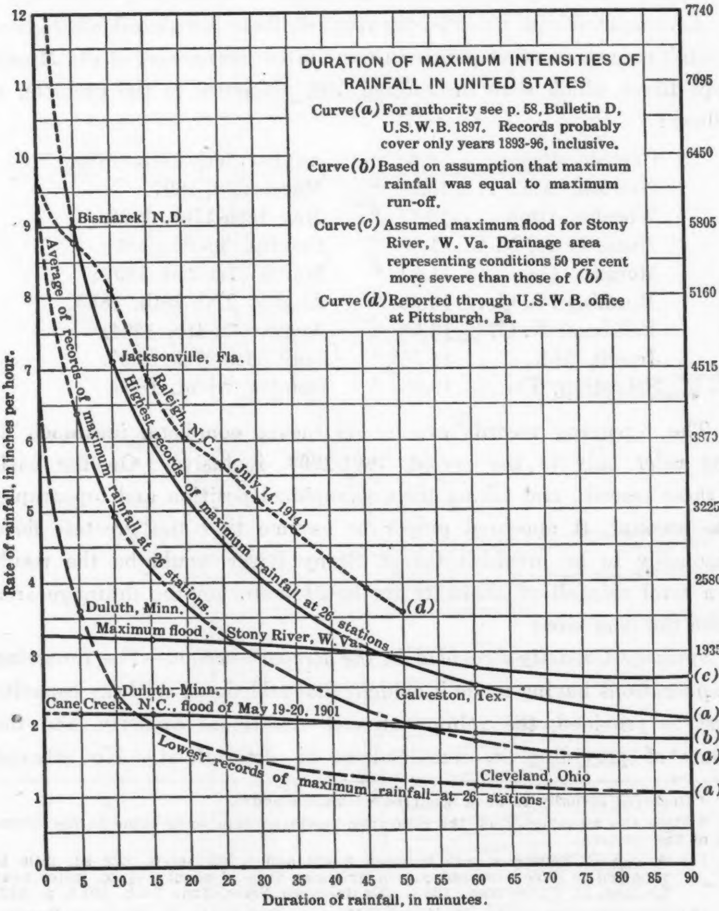



FIG. 9.

or Stony River localities. This conclusion is confirmed by a study of the total rainfall in 24 hours.

The total run-off, as shown in Fig. 12, for the 24-hour period beginning at 7 P. M., May 19th, 1901, is equivalent to a depth of 7.48 in.

over the whole Cane Creek drainage area of 22 sq. miles. For the same period it is reported that records at precipitation stations reasonably near the Cane Creek water-shed show a total of 8 in. of rainfall. (The fact that these two figures check so closely indicates that the estimate of the Cane Creek flood discharge is reasonably accurate.)

As compared with this 24-hour rainfall, there were available 24-hour rainfall records at other points in the United States east of the Mississippi River which were instructive with reference to the problem, as follows:

| | | |
|---|---|---|
| Carlisle, Pa........ | 9.35 in. | August 26th-27th, 1899. |
| Newton, Ala...... | 10.29 " | March 22d, 1897. |
| Wheeler, Ohio..... | 10.47 " | May 16th-17th, 1893. |
| Horse Cove, N. C.. | 11.00 " | October 3d-4th, 1898. |
| Morgan, Ga....... | 11.52 " | March 21st-22d, 1897. |
| Manning, S. C.... | 13.22 " | August 27th-28th, 1893. |
| Falkland, N. C.... | 13.55 " | August 3d-4th, 1894. |
| Jewell, Md........ | 14.75* " | July 26th-27th, 1897. |
| Sebastian, Fla..... | 19.08 " | October 2d-3d, 1899. |

The foregoing records are by no means complete, inasmuch as they refer only to the period, 1891-1899, inclusive. On the basis of these records, and taking the geographical position and topography into account, it appeared proper to assume that the greatest flood reasonably to be provided for at Stony River would be the result of a total rainfall of about 12 in. in 24 hours on the drainage area above the dam site.†

*Spillway Capacity Provided in the Reconstruction.*—The foregoing considerations having made it evident that additional spillway capacity must be provided, the volume of such additional capacity, and the means of providing it, remained to be determined. No attempt

* Reported actually to have occurred within 18 hours.

† Since the reconstruction, the following pertinent data have come to the attention of the writer:

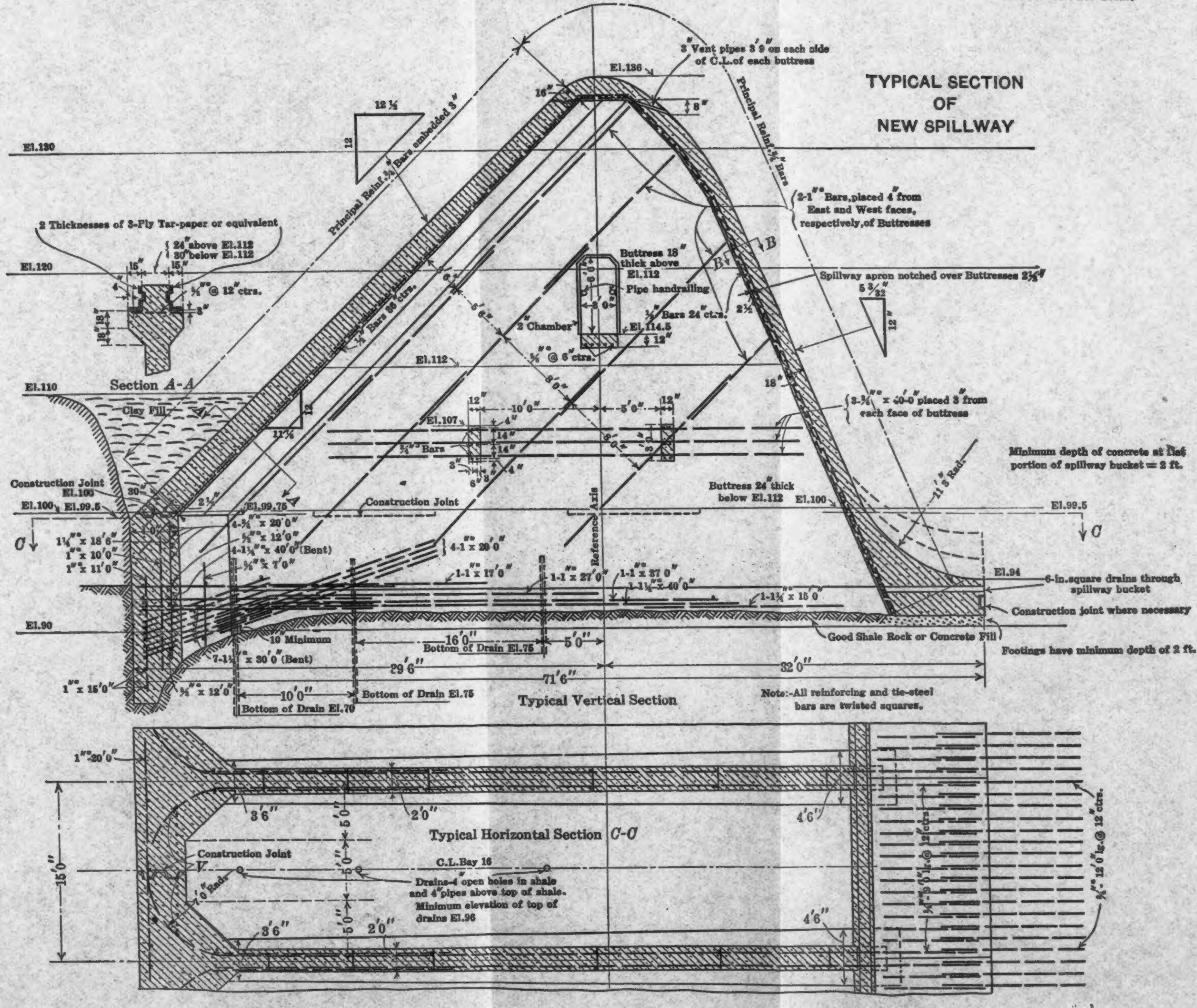
(1) A run-off having a rate between 2 000 and 2 333 sec-ft. per sq. mile is reported to have occurred over a drainage area of about 1½ sq. miles near Le Roy, N. Y., in May, 1916. *Engineering News*, June 24th, 1916, p. 842.

(2) For July 8th, 1916, Professor Alfred J. Henry, U. S. Weather Bureau, reports that the area (Alabama and Georgia) covered by a 24-hour rainfall of 8 in. and more was about 4 945 sq. miles. *Monthly Weather Review*, August, 1916, p. 467.

(3) A 24-hour rainfall of 22.22 in. is reported to have occurred on July 15th-16th, 1916, at Altapass, Mitchell County, N. C. (Elevation 2 625). *Ibid*, p. 467.

(4) It is reported that rain fell at the rate of 5.21 in. per hour for 25 min. at Mobile, Ala., on July 8th, 1916. *Ibid*, p. 468.

PLATE III.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
SCHEIDENHELM ON
RECONSTRUCTION OF
STONY RIVER DAM.



will be made in this paper to discuss the alternatives considered for increasing the original spillway capacity of the dam, for such alternatives differed essentially only as to the extent to which the several available factors were utilized. Instead, the conditions caused by the maximum flood, within the limits of reason, will be applied to the means finally adopted for increasing the spillway capacity, in order to test the adequacy of these means. The adopted spillway provision, which will be understood by reference to Plate II, was obtained by the following means:

1.—Reconstructing that portion of the original bulkhead section of the dam between Buttresses 10 and 19 to form a second spillway. The transverse cross-section of this new spillway is shown in Plate III. It was provided with a suitable reinforced concrete channel mat extending about 85 ft. down stream from the bucket of the spillway apron.

2.—Adding 3 ft. 6 in. to the height of the original bulkhead portions of the dam outside of the spillways, *viz.*, east of the old spillway and west of the new spillway. This was accomplished by adding a parapet, of the form shown in dotted outline in Plate V, with its top at Elevation 142.5.

3.—Increasing the height of that portion of the original bulkhead section of the dam between Buttresses 19 and *A* (*viz.*, between the old and new spillways) by the addition of a parapet 1 ft. high. This forms an intermediate spillway with its crest at Elevation 140, as shown in the typical sections of Plate V.

4.—Utilizing the horizontal upper member of the new anchoring wall at the toe of the dam, between the old and new spillways, to form a reinforced concrete mat or "tumbling hearth" to receive the impact of the sheet of water falling well-nigh vertically from the intermediate spillway. This horizontal mat, together with an inclined, reinforced concrete, protection mat (section *B-B* of Plate V), forms a channel sloping downward from the ends of the intermediate spillway section toward, and discharging through, a new outlet channel which extends down stream from Gate-bays 30 and 31.

5.—Improving the shape of the crest of the original or "old" spillway. This warranted the assumption of a somewhat higher coefficient of discharge, as applied in the Francis formula.

Under the original conditions, disregarding wave action, overtopping of the dam would have occurred when the water level in the reservoir had reached Elevation 139; whereas, under the conditions obtaining after reconstruction, overtopping cannot occur until the water level reaches Elevation 142.5. It is probable that overtopping due solely to wave action would be of minor consequence. However, as will presently be shown, the water level due to the greatest flood reasonably to be provided for would probably leave a margin of about 6 in. below Elevation 142.5.

*Maximum Flood Within Limits of Reason.*—For the purpose of studying the effect of such a flood, in the case of the Stony River Dam and Reservoir, let it be assumed that it would have a graph similar to that shown in Fig. 13. This graph was derived by increasing the rates of run-off shown in Fig. 12 by 50%, thus corresponding to the ratio of the estimated maximum 24-hour precipitation to the 8-in. precipitation reported to have occurred within a like period in the case of the Cane Creek flood. It is true, of course, that no two rain storms or floods are exactly alike; yet, for the purpose in mind, the foregoing assumption appeared to afford a reasonable basis for design.

For purposes of comparison, there has been plotted on Fig. 9 a curve (*c*) showing the rainfall for the critical 90-min. period of such assumed greatest possible flood discharge, the curve being based, as before, on the assumption (not strictly true) that the rate of rainfall would be equal to the rate of run-off. Even allowing for the evident equalization, or smoothing-off of the peak of the rainfall, it is apparent that a rainfall causing a flood such as that represented by the curve (*c*) would not equal certain actual records of rainfall.

*Equalization Effect of Reservoir.*—In studying the results of such floods as those plotted on Figs. 12 and 13, when applied to the Stony River drainage area and spillway provision, it may reasonably be assumed that in each instance there would be one point of time when, for all practical purposes, a state of equilibrium would exist. At such

a time the total run-off from the drainage area and the discharge over the spillway would be equal. Thereafter, as the run-off increased, a certain quantity of water would be stored over the whole reservoir area for each foot of rise in water level. The extent of such equalization or absorption would depend on the spillway capacity and the reservoir area.

The capacity of the reservoir above the elevation of the lower spillway crests is shown on Fig. 10. On Fig. 11 is plotted the spillway capacity (after reconstruction), derived from the data and assumptions there shown.

On the basis of these diagrams, the equalization effect of the reservoir has been studied with reference to a number of flood conditions, as follows:

I.—Before considering the greatest flood reasonably to be anticipated, it appears proper to study the equalization effect of the reservoir on a flood exactly similar, and equal in intensity, to that reported for Cane Creek on May 19th-20th, 1901. The result of such study is shown in Fig. 12. The spillway crests are assumed to be without flash-boards.

The method of deriving the curves showing the discharge over the spillways is as follows: Referring to Fig. 12, it is assumed that at 7 A. M. the total run-off from the drainage area and the discharge over the spillways are equal. The graph of the flood then shows that at this time the run-off would be approximately 100 cu. ft. per sec., which (from Fig. 11) would require the reservoir water level to be at Elevation 136.2. The effect of the storage capacity of the reservoir at increasing elevations of head-water is then determined at arbitrary intervals, depending on the refinement desired. Selecting a 0.3-ft. rise in water level as the first interval, it is found from the reservoir capacity curve of Fig. 10 that for this interval the reservoir will absorb approximately 39 000 000 gal. This quantity, when converted into the proper units and scale, is represented in Fig. 12 by the area, *a-b-c, viz.*: The point, *b,* of course, must have an ordinate representing the spillway discharge at Elevation 136.5, approximately 365 cu. ft. per sec. (from Fig. 11). The position of the line, *b-c,* is then determined by trial, the required area, *a-b-c,* having been determined as above stated.

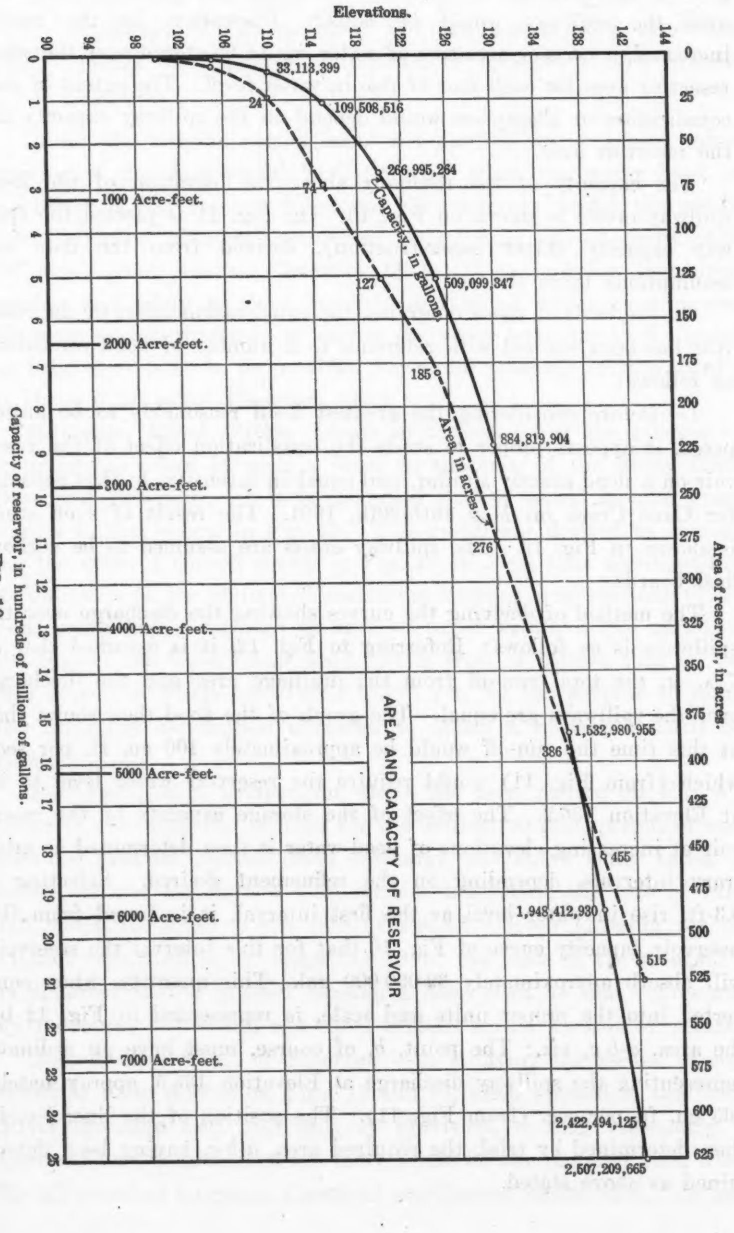

Elevations.
90 94 98 102 106 110 114 118 122 126 130 134 138 142 144
Area of reservoir, in acres.
0 25 50 75 100 125 150 175 200 225 250 275 300 325 350 375 400 425 450 475 500 525 550 575 600 625
Capacity of reservoir, in hundreds of millions of gallons.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25
1000 Acre-feet.
2000 Acre-feet.
3000 Acre-feet.
4000 Acre-feet.
5000 Acre-feet.
6000 Acre-feet.
7000 Acre-feet.
Capacity in gallons.
Area in acres.
33,113,399
109,508,516
266,995,264
509,099,347
884,819,904
1,532,980,955
1,948,412,930
2,422,494,125
2,507,209,665
24
74
127
185
276
386
455
515
AREA AND CAPACITY OF RESERVOIR

FIG. 10.

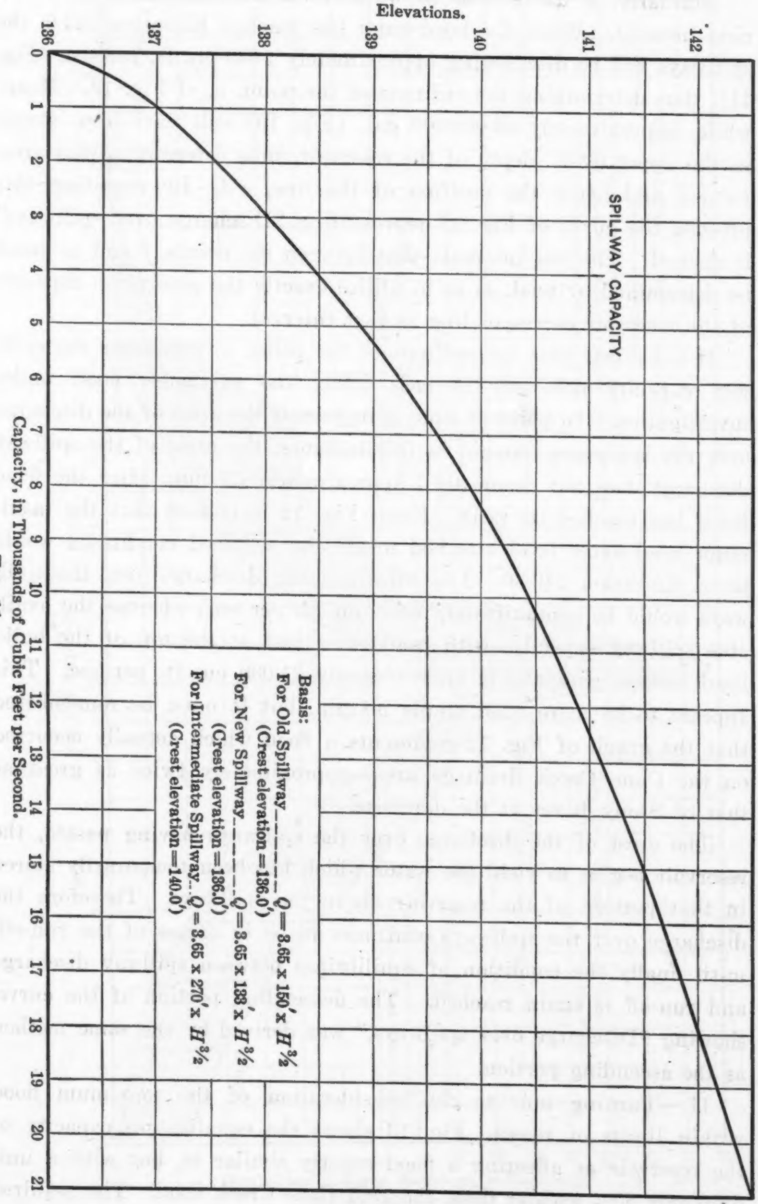

Elevations.
136
137
138
139
140
141
142
SPILLWAY CAPACITY
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
Capacity, in Thousands of Cubic Feet per Second.
Basis:
For Old Spillway ... $Q = 3.65 \times 150' \times H^{3/2}$
(Crest elevation = 136.0')
For New Spillway ... $Q = 3.65 \times 138' \times H^{3/2}$
(Crest elevation = 136.0')
For Intermediate Spillway ... $Q = 3.65 \times 270' \times H^{3/2}$
(Crest elevation = 140.0')

FIG. 11.

Similarly, a 0.5-ft. rise in water level may be selected for the next interval. When the head-water has reached Elevation 137.0, the spillways will be discharging approximately 1 030 cu. ft. per sec. (Fig. 11), thus determining the ordinate of the point, *e*, of Fig. 12. Meanwhile, approximately 66 000 000 gal. (Fig. 10) will have been stored in the upper 0.5-ft. depth of the reservoir, thus determining the area, *b-c-d-e*, and hence the position of the line, *e-d*. By repeating this process, the curve of Fig. 12 representing "Discharge over spillways" is defined. The last interval—that between the points, *f* and *g*—must be determined by trial, so as to utilize exactly the absorption capacity of the reservoir corresponding to that interval.

It is evident that the ordinate of the point, *g*, represents the spillway capacity necessary to pass safely the particular flood under investigation. In point of time, *g* represents the crest of the discharge over the spillways, though, in this instance, the crest of the spillway discharge does not occur until approximately 35 min. after the flood itself has reached its peak. From Fig. 12 it is seen that the maximum head-water level attained under the assumed conditions would be at Elevation 140.36. The corresponding discharge over the spillways would be approximately 9 600 cu. ft. per sec., whereas the available spillway capacity, with head-water just at the top of the bulkhead section parapets, is approximately 21 000 cu. ft. per sec. This appears to be more than ample margin, but it must be remembered that the graph of Fig. 12 represents a flood which actually occurred on the Cane Creek drainage area—approximately twice as great as that of Stony River at the dam site.

The crest of the discharge over the spillways having passed, the reservoir begins to yield the water which has been temporarily stored in that portion of the reservoir above the spillways. Therefore the discharge over the spillways continues to be in excess of the run-off, until finally the condition of equilibrium between spillway discharge and run-off is again reached. The descending portion of the curve, showing "Discharge over spillways," was derived by the same method as the ascending portion.

II.—Turning now to the consideration of the maximum flood within limits of reason, Fig. 13 shows the equalization capacity of the reservoir as affecting a flood exactly similar to, but with a unit discharge 50% greater than, the 1901 Cane Creek flood. The required

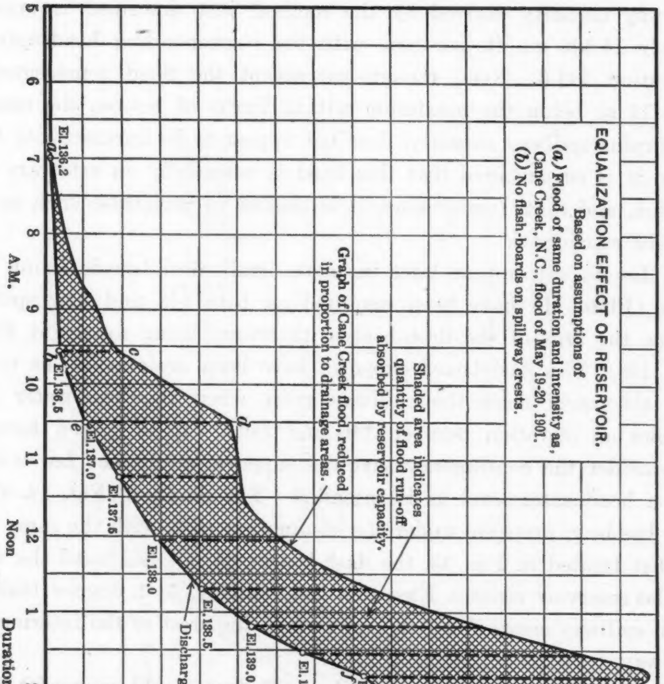

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a.) Flood of same duration and intensity as Cane Creek, N.C., flood of May 19-20, 1901.
(b.) No flash-boards on spillway crests.
Cane Creek drainage area=22 sq. miles
Stony River " " =11.4 sq. miles.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Graph of Cane Creek flood, reduced in proportion to drainage areas.
Required spillway capacity, 9,600 cu. ft. per sec.
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
Discharge over spillways.
El.136.2
El.136.5
El.137.0
El.137.5
El.138.0
El.138.5
El.139.0
El.139.5
El.140.0
El.140.36
El.140.0
El.139.5
El.139.0
El.138.5
El.138.0
El.137.5
A.M.
Noon
P.M.
Duration of Flood
Run-off and spillway discharge, in thousands of cubic feet per second.

FIG. 12.

spillway capacity derived by the method just described is approximately 15 600 cu. ft. per sec., with the corresponding head-water at Elevation 141.6. Even though we accept the flood represented in Fig. 13 as being the maximum within limits of reason, the margin, or surplus spillway capacity, does not appear to be unreasonably large when it is remembered that this flood is essentially an arbitrary conception, and that, furthermore, it is subject to variations such as will now be considered.

III.—As will appear later in more detail, flash-boards about 3 ft. high (Plate II) have been mounted on both old and new spillway crests, the tops of the flash-boards, therefore, being nearly at Elevation 139. The flash-board supports have been designed so as to fail and absolutely clear the spillway crest when the head-water level reaches an elevation between 140 and 140.5. It is proper, however, to consider the contingency that the supports might not fail with so low a head-water level as is intended. The diagram, Fig. 14, therefore has been prepared under the assumption that, with the same flood as that treated in Fig. 13, the flash-boards do not fail until the water in the reservoir reaches Elevation 141, namely, 5 ft. higher than the main spillway crests and 1 ft. higher than the crest of the intermediate spillway.

In this case the initial state of equilibrium would exist with head-water at about Elevation 139.3, that is, 0.3 ft. above the assumed top of the flash-boards. The effect of the initial rise of head-water (to Elevation 141.0) is determined in the same manner as described previously. With the breaking of the flash-board supports, however, and the consequent sweeping away of the flash-boards when Elevation 141 is reached, the discharging capacity of the spillways is suddenly increased—as is evident from the diagram—to such an extent that, temporarily, the discharge over the spillways is greater than the run-off from the drainage area. This, of course, would not necessarily be the case with a flood having a graph of different shape. The position of the point, *c*, was determined by trial, the area, *a-b-c*, representing the quantity of water stored in the reservoir in the depth interval between points, *c* and *a*, because this quantity must be yielded by the reservoir before the head-water can recede to approximately Elevation 140.52 at the point, *c*. Actually, *a-c*, would undoubtedly not be a straight line; a more accurate location would be obtained by

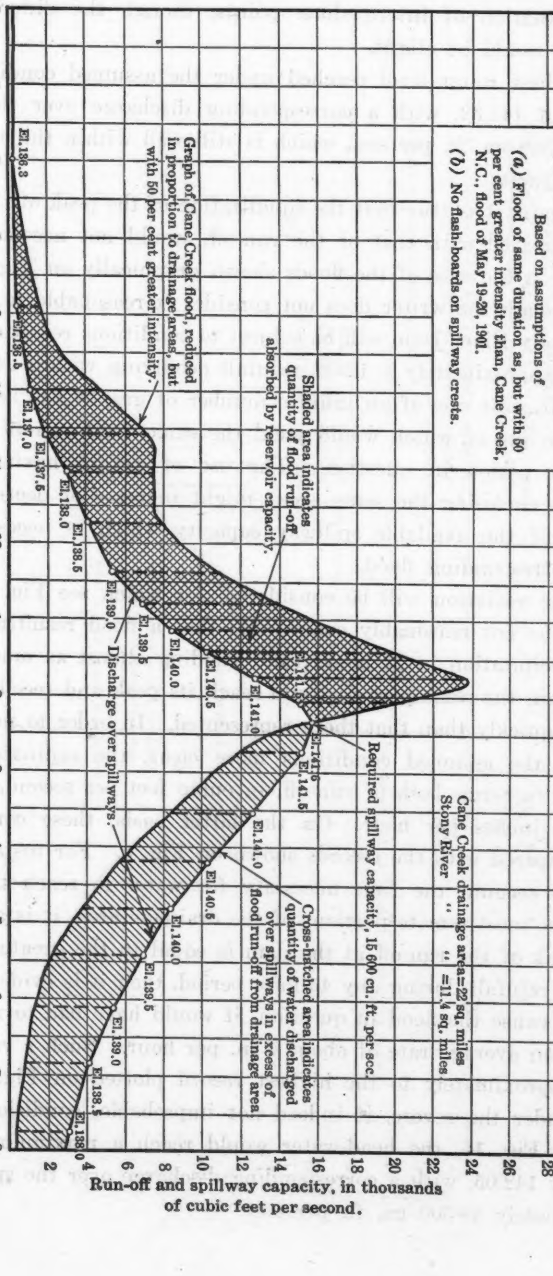

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Flood of same duration as, but with 50 per cent greater intensity than, Cane Creek, N.C., flood of May 19-20, 1901.
(b) No flash-boards on spillway crests.
Cane Creek drainage area=22 sq. miles
Stony River " " =11.4 sq. miles
Required spillway capacity, 15 600 cu. ft. per sec.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
Graph of Cane Creek flood, reduced in proportion to drainage areas, but with 50 per cent greater intensity.
Discharge over spillways.
El.136.3
El.136.5
El.137.0
El.137.5
El.138.0
El.138.5
El.139.0
El.139.5
El.140.0
El.140.5
El.141.0
El.141.5
El.141.6
El.141.0
El.140.5
El.140.0
El.139.5
El.139.0
El.138.5
El.138.0
5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8
A.M.
Noon
P.M.
Duration of Flood.
2 4 6 8 10 12 14 16 18 20 22 24 26 28
Run-off and spillway capacity, in thousands of cubic feet per second.

FIG. 13.

the determination of intermediate points, though the difference in final result would be slight.

The highest water level reached under the assumed conditions is at Elevation 141.92, with a corresponding discharge over the spillways of 17 500 cu. ft. per sec., which is still well within the available spillway capacity.

IV.—Again, it is true that the equalization of the peak of the rainfall, as compared with that of the run-off, would not necessarily be as great as in the case of the floods shown graphically on Figs. 12 to 14; and, though the writer does not consider it reasonable to assume that the Stony River Dam will be subject to conditions resulting from more than approximately a 12-in. rainfall occurring within 24 hours, yet, thus far, only one of an infinite number of graphs has been considered, any one of which would yield the same total run-off during the 24-hour period in question. The use of graphs having other shapes (but enclosing the same area) might increase or decrease the proportion of the available spillway capacity which is necessary to pass the corresponding flood.

Only one variation will be considered, for which see Fig. 15. It appears to be not reasonably conceivable that a flood resulting from a 12-in. precipitation in 24 hours (and yielding almost as much total run-off within the same period) could reach its peak and recede therefrom more quickly than that there represented. In order to make the severity of the assumed conditions more clear, the ordinates have been shown in terms both of run-off, in cubic feet per second, and of rainfall, in inches per hour. On the latter basis, these conditions may be compared with the records shown on Fig. 9. For instance, if, taking into account the time necessary for water to reach the dam site from the most remote portions of the drainage area, it is assumed that the peak of the run-off at the dam is equal to the greatest average rate of rainfall during any 45-min. period, then it is evident that, in order to cause the flood in question, it would have had to rain for 45 min. at an average rate of about 4 in. per hour. Such a rate corresponds approximately to the highest record plotted on Fig. 9.

Even under the severe, if indeed not improbable, conditions represented in Fig. 15, the head-water would reach a maximum elevation of only 142.05, with a corresponding discharge over the spillways of approximately 18 300 cu. ft. per sec.

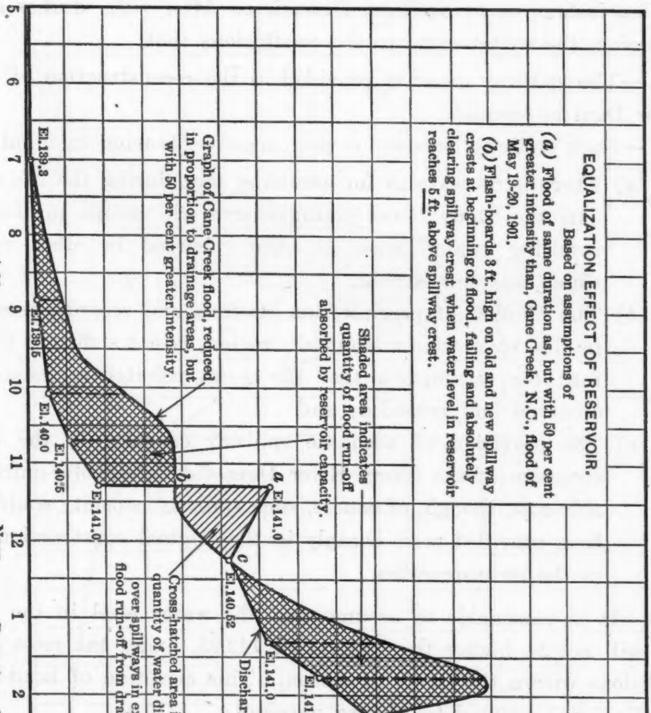

EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Flood of same duration as, but with 50 per cent greater intensity than, Cane Creek, N.C., flood of May 19-20, 1901.
(b) Flash-boards 3 ft. high on old and new spillway crests at beginning of flood, failing and absolutely clearing spillway crest when water level in reservoir reaches 5 ft. above spillway crest.
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Graph of Cane Creek flood, reduced in proportion to drainage areas, but with 50 per cent greater intensity.
Cross-hatched area indicates quantity of water discharged over spillways in excess of flood run-off from drainage area.
Discharge over spillways.
Required spillway capacity 17 500 cu. ft. per sec.
El.139.3
El.139.6
El.140.0
El.140.5
El.141.0
a El.141.0
b
c
El.140.62
El.141.0
El.141.5
El.141.92
5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8
A.M.
Noon
P.M.
Duration of Flood.
0 2 4 6 8 10 12 14 16 18 20 22 24 26
Run-off and spillway discharge, in thousands of cubic feet per second.

FIG. 14.

*Conclusions as to Spillway Provision.*—After such studies as the foregoing, the writer came to the conclusions that:

1.—The spillway capacity provided in the reconstruction of Stony River Dam is adequate.

2.—Such spillway capacity is not excessive, bearing in mind that:

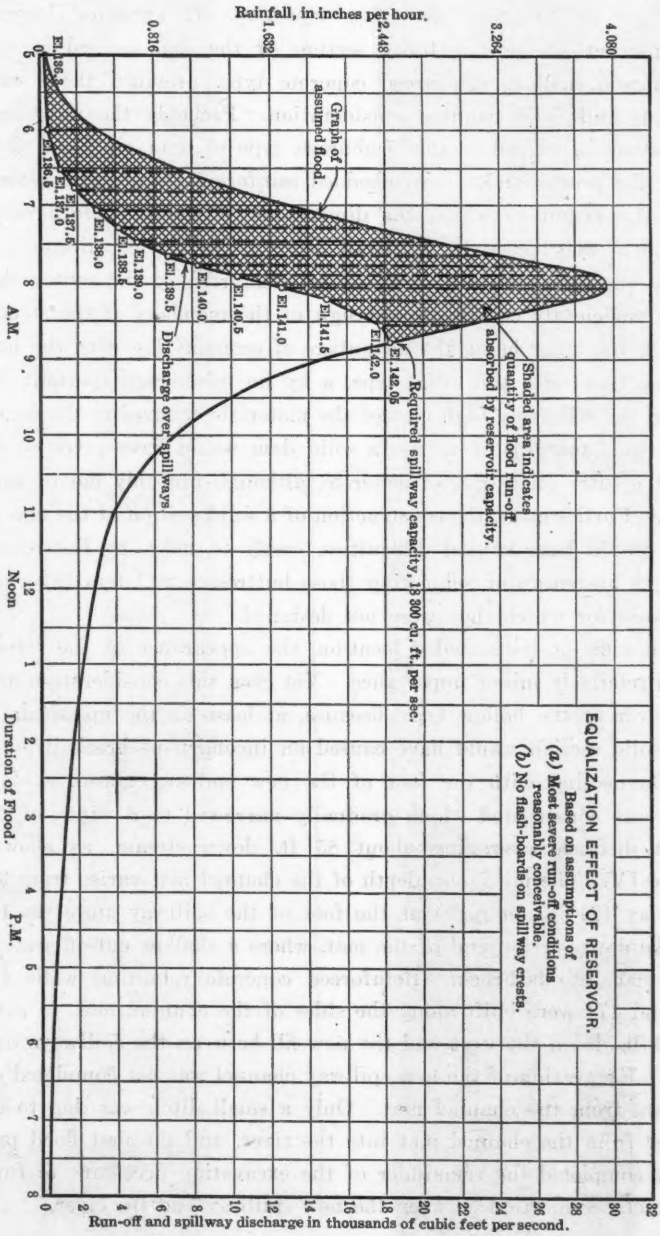
(*a*) There is no warrant for assuming that during the life of the dam the Stony River drainage area will escape rainfall and resulting run-off such as have occurred in other regions which are comparable;

(*b*) The number of precipitation stations and records, especially in the Appalachian Mountain region, is not sufficient to warrant the assumption that the worst possibilities have been observed and recorded; and

(*c*) The provision of adequate spillway capacity in the reconstruction of the Stony River Dam did not involve unreasonable cost, though, of course, such spillway capacity could have been provided more cheaply in the original construction than in the reconstruction.

3.—It is reasonable to assume that the water level in the reservoir will not be higher than Elevation 142.25, under the most severe conditions within the limits of reason. This elevation of head-water, therefore, was adopted for design purposes.

4.—Granting the necessity for making provision to pass safely the greatest flood reasonably to be anticipated, it is also true that such a flood would be far from normal. Hence, in considering the "normal maximum load" to which the dam will be subjected, it appeared proper to assume the head-water at Elevation 140.5. At this elevation the flash-board supports (on the old and new spillways) are reasonably certain to have failed. The flash-boards and supports having thus been swept from the main spillway crests, the spillway capacity would be approximately 10 000 cu. ft. per sec., without taking into consideration any equalizing effect of the reservoir. In the case of the original structure, the spillway capacity likewise was approximately 10 000 cu. ft. per sec. at Elevation 140.5, it being noted, however, that such head-water elevation could not have existed without causing the original bulkhead sections (with crest at Elevation 139) to be overtopped. The corresponding unit spillway capacity is about 875 cu. ft. per sec. per sq. mile of drainage area.



Rainfall, in inches per hour.
0.816
1.632
2.448
3.264
4.080
EQUALIZATION EFFECT OF RESERVOIR.
Based on assumptions of
(a) Most severe run-off conditions reasonably conceivable.
(b) No flash-boards on spillway crests.
Graph of assumed flood
Shaded area indicates quantity of flood run-off absorbed by reservoir capacity.
Required spillway capacity, 18 300 cu. ft. per sec.
Discharge over spillways.
El.136.5
El.136.5
El.137.0
El.137.5
El.138.0
El.138.5
El.139.0
El.139.5
El.140.0
El.140.5
El.141.0
El.141.5
El.142.0
El.142.05
A.M.
Noon
P.M.
Duration of Flood
Run-off and spillway discharge in thousands of cubic feet per second.


FIG. 15.

*Type of Structure for New Spillway.*—It appeared logical to reconstruct the new spillway section of the dam according to the Ambursen, hollow, reinforced concrete type, provided there was no serious and valid counter consideration. Probably the most serious questions in regard to the Ambursen type of dam are those dealing with the protection of the embedded reinforcing steel from corrosion, and the extent to which the dimensions of the reinforced concrete members may properly be pared down to theoretical limits. Both these questions may be answered satisfactorily, so the writer thinks, by a sufficiently conservative design of the members of the structure.

On the other hand, the advantage of economy lay with the hollow, rather than with the solid, type, a by no means unimportant factor being the relatively high cost of the materials composing the concrete. For equal margins of safety, a solid dam would have involved somewhat greater quantities of concrete, although probably not of excavation. Furthermore, the construction of a solid section at the new spillway would have caused difficulties, costly to meet, at Buttresses 10 and 19 by reason of subjecting those buttresses to lateral hydrostatic pressure for which they were not designed.

In view of its secluded location, the appearance of the structure is of relatively minor importance. Yet even this consideration argued in favor of the hollow type, because, at least on the up-stream side, the solid section would have caused an incongruous break in outline.

Connecting with the foot of the new spillway apron, a channel mat was constructed which gradually narrowed to a width of 60 ft. at a distance averaging about 85 ft. down stream, as shown in Plate IV. The minimum depth of the channel mat varies from 30 in. on clay (24 in. on rock) at the foot of the spillway apron to 18 in. at the down-stream end of the mat, where a shallow cut-off wall seals the mat into bed-rock. Reinforced concrete retaining walls (Figs. 16 and 17) were built along the sides of the channel mat, to support the hillside on the west and the new fill between the spillways on the east. Excavation of the new spillway channel was not completed down stream from the channel mat. Only a small ditch was dug to drain water from the channel mat into the river, and the first flood practically completed the remainder of the excavation necessary to furnish sufficient connection between the new spillway and the river.

STONY RIVER DAM AFTER RECONSTRUCTION.

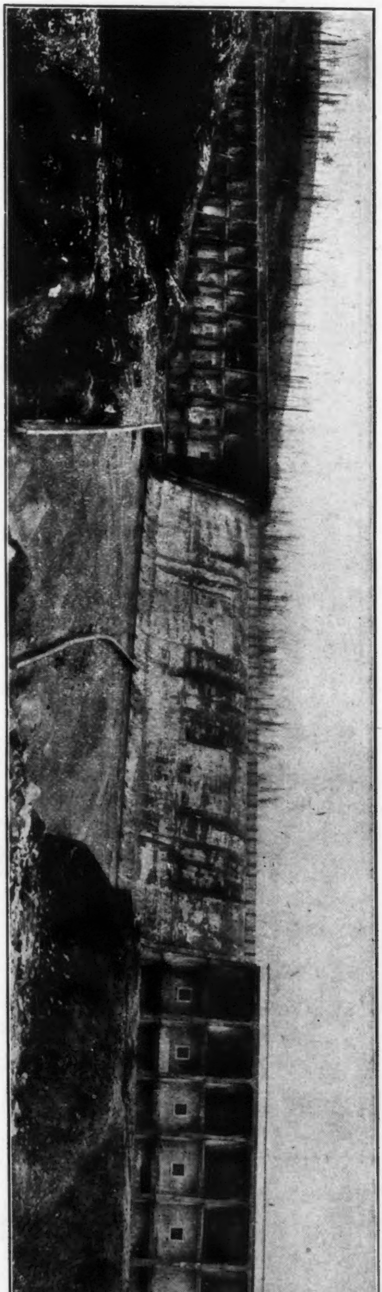


FIG. 16.—EASTERN PART, SHOWING OLD SPILLWAY.

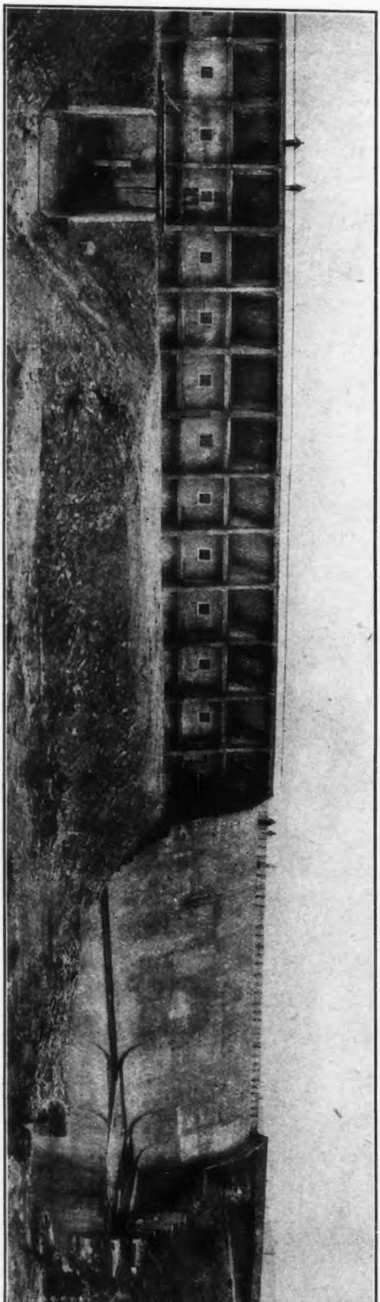
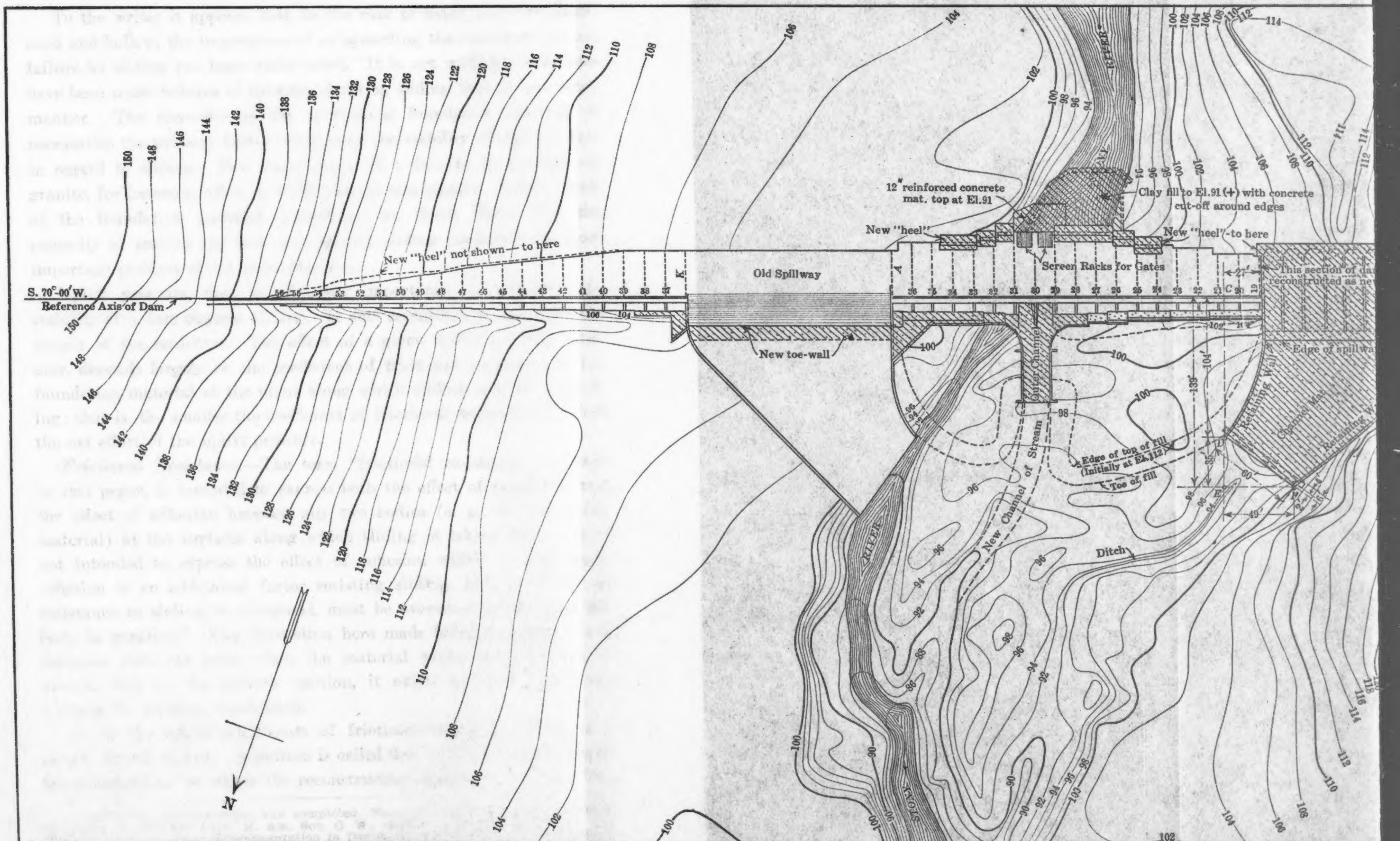
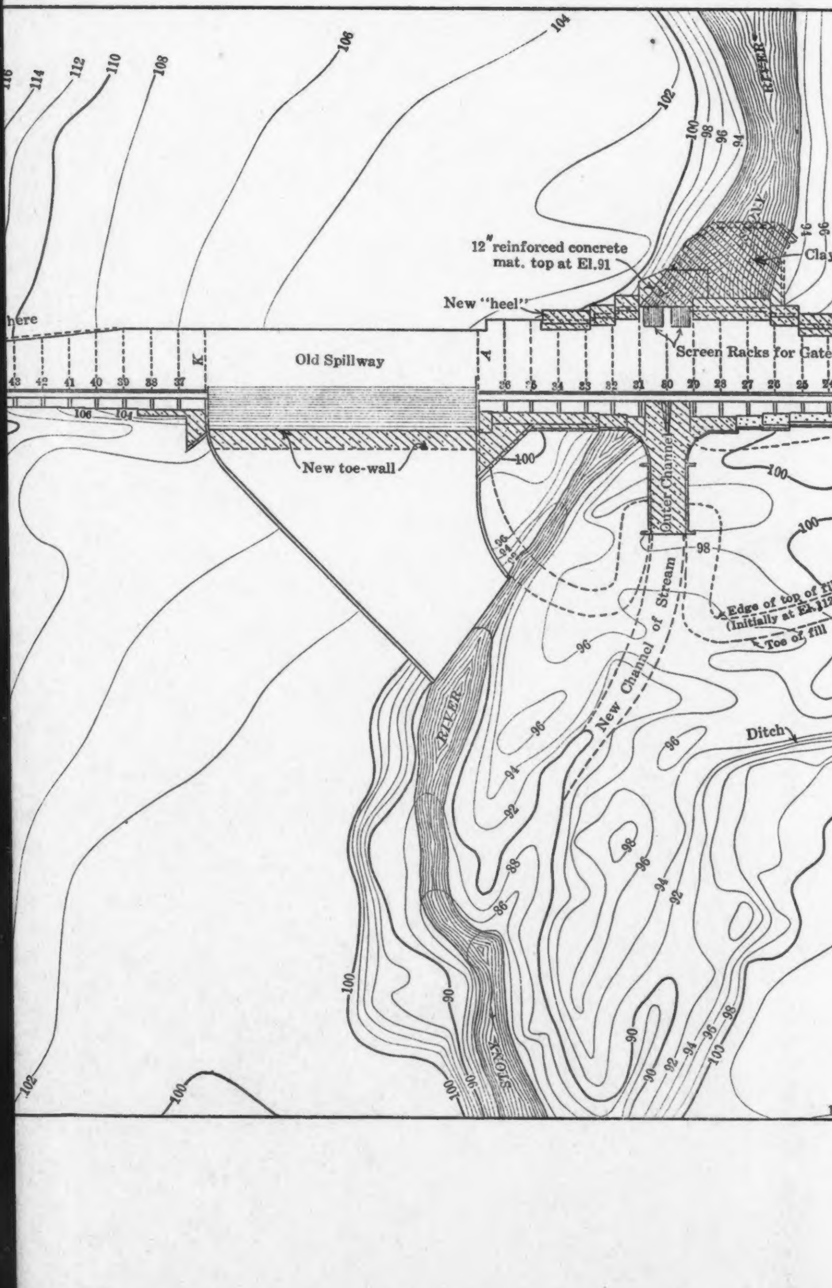


FIG. 17.—WESTERN PART, SHOWING NEW SPILLWAY.

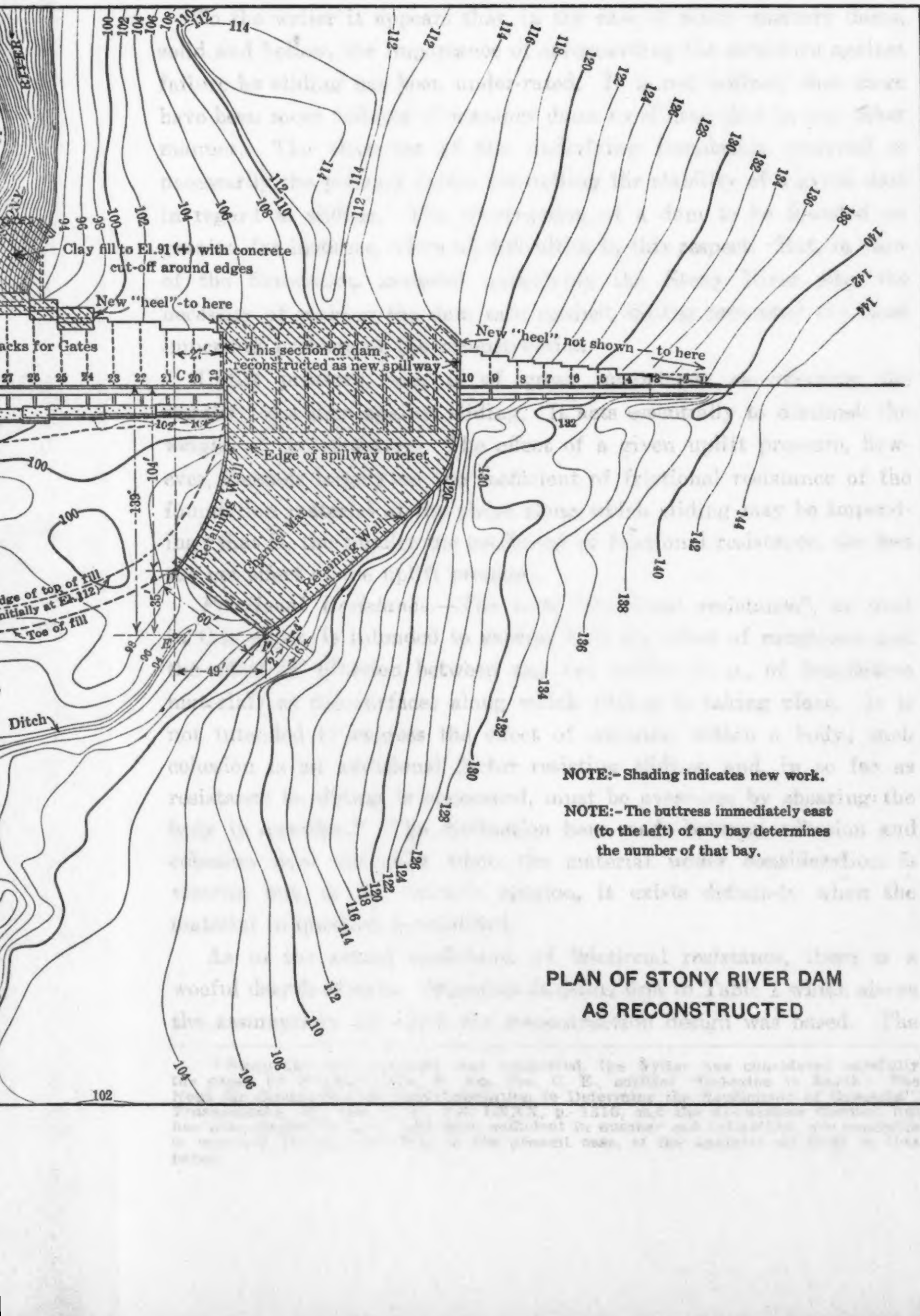


New "heel" not shown — to here
Old Spillway
S. 70°-00' W.
Reference Axis of Dam
New toe-wall
12" reinforced concrete mat. top at El.91
New "heel"
Screen Racks for Gates
Clay fill to El.91 (+) with concrete cut-off around edges
New "heel"-to here
Edge of top of fill (Initially at El.112)
Toe of fill
Ditch
New Channel of Stream
RIVER
N



12" reinforced concrete mat. top at El.91
New "heel"
Old Spillway
Screen Racks for Gate
New toe-wall
Outer Channel
New Channel of Stream
Edge of top of fi
(Initially at El.112
Toe of fill
Ditch
RIVER
STONY

PLATE IV.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
SCHEIDENHELM ON
RECONSTRUCTION OF
STONY RIVER DAM.




PLAN OF STONY RIVER DAM
AS RECONSTRUCTED

## Resistance to Sliding.

To the writer it appears that, in the case of many masonry dams, solid and hollow, the importance of safeguarding the structure against failure by sliding has been under-rated. It is not unlikely that there have been more failures of masonry dams by sliding that in any other manner. The character of the underlying foundation material is necessarily the primary factor controlling the stability of a given dam in regard to sliding. The construction of a dam to be founded on granite, for instance, offers no difficulties in this respect. But, in view of the foundation material underlying the Stony River site, the necessity of making the dam safe against sliding presented the most important problem of the reconstruction.

Uplift pressure, too, is of great importance as affecting the stability of a dam against sliding. It acts essentially to diminish the weight of the structure. The effect of a given uplift pressure, however, depends largely on the coefficient of frictional resistance of the foundation material at the plane along which sliding may be impending; that is, the smaller the coefficient of frictional resistance, the less the net effect of the uplift pressure.

*Frictional Resistance.*—The term "frictional resistance", as used in this paper, is intended to express both the effect of roughness and the effect of adhesion between any two bodies (*e. g.*, of foundation material) at the surfaces along which sliding is taking place. It is not intended to express the effect of cohesion within a body; such cohesion is an additional factor resisting sliding, and, in so far as resistance to sliding is concerned, must be overcome by shearing the body in question.* The distinction here made between adhesion and cohesion does not exist when the material under consideration is viscous, but, in the writer's opinion, it exists definitely when the material in question is solidified.

As to the actual coefficients of frictional resistance, there is a woeful dearth of data. Attention is called first to Table 1 which shows the assumptions on which the reconstruction design was based. The

* Since the reconstruction was completed, the writer has considered carefully the paper by William Cain, M. Am. Soc. C. E., entitled "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion", *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 1315, and the discussions thereon, but has been unable to agree that data, sufficient in number and reliability, are available to warrant the use, especially in the present case, of the analysis set forth in that paper.

TABLE 1.—COEFFICIENTS OF FRICTIONAL RESISTANCE FORMING THE BASIS OF DESIGN IN THE RECONSTRUCTION OF STONY RIVER DAM.

| Character of foundation material. | Value assumed for design. | Probable minimum value. | Ratio. |
|---|---|---|---|
| Clayey overburden—wet. Applying to footings of the original sections of dam and to probable "planes of least resistance" in clayey foundation soil. | 0.33 | 0.25 | 0.75 |
| Shale with clay seams. Applying to shale near surface of bed-rock; hence including shale immediately underlying new footings at Buttresses 10 to 19, inclusive (new spillway section.) | 0.40 | 0.25 | 0.625 |
| Hard shale, not disintegrated. Applying to shale at probable "planes of least resistance" at Buttresses 10 to 19, inclusive (new spillway section.) | 0.50 | 0.40 | 0.80 |

"planes of least resistance" referred to in that table are explained later.

The pertinent data considered in adopting these assumptions follow.

*Frictional Resistance of Clayey Soil.*—In the writer's opinion, the safe coefficient of frictional resistance for the clayey foundation soil at the dam site when wet is between 0.3 and 0.4. When such soil is dry or merely moist, the frictional resistance, of course, is greater than when it is thoroughly wet; but it is not safe to assume that the foundation soil will always be fairly dry. It is true that the leakage from the reservoir into the foundation soil has, at this writing, become, and in all probability will continue to be, a relatively minor factor. However, ground-water in the soil cannot be disposed of, especially inasmuch as the dam site receives the ground-water drainage of the adjoining hillsides. The new drainage system of the dam cannot cause the foundation soil to become dry. It can merely prevent the accumulation of serious uplift pressure.

The results of various tests of frictional resistance are of record. Unfortunately, they are not numerous; nor are the test conditions sufficiently standardized, nor the character of the material under tests sufficiently well defined, to allow satisfactory detailed comparison with each other or with the conditions to which it is attempted to apply their results. Moreover, it is the rule, rather than the exception, that, as in the case in point, the foundation material is heterogeneous, thus increasing the difficulty of drawing comparisons with previous test

results and making more necessary the use of judgment in selecting coefficients for design or other purposes.

Among the recorded data are the experiments of General Morin, as referred to by Rankine,* which indicate a coefficient of frictional resistance of 0.31 for wet clay on wet clay, as tested by him. Tests of the foundation soil in question, however, are more pertinent, and such tests were made on several kinds of soil by Mr. D. N. Showalter, Resident Engineer, under the direction of the writer. The results of these tests are given in Table 2.

In the ordinary case, these tests were made by tamping a given kind of soil into a box having an open top, the box being a cube of about 12 in., then capping the boxful of soil with a solid piece of the same material, so that it protruded above the edge of the box, and turning the box upside down. The protruding material, after having been given an approximately horizontal surface and made smooth, was utilized as the moving element in determining the frictional resistance on a prepared, smooth, horizontal bed of the same kind of material in place. The moving element was drawn along by hand over the bed, the required pull being measured by a spring balance. Ordinarily, more than ½ sq. ft. of the material under test was actually in contact. The method of making the tests is illustrated by Figs. 8 and 18.

In the case of the tests made in cold weather, the clay bed had been scraped previously so as to remove all frozen material and any "mud" formed by melting snow. When not under test, the surface or bed was protected by a covering of tar-paper. In the last series of tests, in May, 1915, a galvanized-iron pail was used instead of a wooden box. The result was to make these experiments appear even more crude than was really the case; but, inasmuch as the weight of the moving element and the required pull were not less readily determinable, such experiments are not believed to be the less worthy of consideration.

Despite the absence of laboratory refinements, the results obtained are in all probability reasonably indicative of the actual frictional resistance, as previously defined. Among the results of any single group of experiments, the maximum variation from the mean was 30% of the mean, and the average of such maximum variation for all

* "A Manual of Applied Mechanics", p. 211.

TABLE 2.—RESULTS OF TESTS OF FRICTIONAL RESISTANCE OF FOUNDATION SOIL AT STONY RIVER DAM SITE.

| Experiment No. | Character of soil under test, | Condition of surfaces in contact. | Temperature range, in degrees Fahrenheit. | Load, in pounds. | Pull (while in motion), in pounds. | Coefficient of frictional resistance. |
|---|---|---|---|---|---|---|
| 1.... | White clay, fine-grained.......... | Wet. | 35 to 41 | 160 | 64 | 0.40 |
| 2.... | do. | do. | do. | 160 | 67 | 0.42 |
| 3.... | do. | do. | do. | 284 | 80 | 0.28 |
| 4.... | do. | do. | do. | 284 | 72 | 0.25 |
| 5.... | do. | do. | do. | 160 | 55 | 0.34 |
| | | | | | Average | 0.34 |
| 6.... | White clay, fine-grained........... | Moist. | 35 to 41 | 160 | 75 | 0.47 |
| 7.... | do. | do. | do. | 160 | 72 | 0.45 |
| 8.... | do. | do. | do. | 284 | 118 | 0.42 |
| 9.... | do. | do. | do. | 220 | 78 | 0.35 |
| 10.... | do. | do. | do. | 220 | 85 | 0.39 |
| | | | | | Average | 0.42 |
| 11.... | Yellow clay, containing some grit. | Fairly wet. | 20 to 25 | 149 | 120 | 0.80 |
| 12.... | do. | do. | Soil was thawed by a fire near-by. | 149 | 120 | 0.80 |
| 13.... | do. | do. | | 149 | 106 | 0.71 |
| 14.... | do. | do. | | 149 | 114 | 0.76 |
| 15.... | do. | do. | | 189 | 110 | 0.58 |
| 16.... | do. | do. | | 189 | 99 | 0.52 |
| 17.... | do. | do. | | 249 | 120 | 0.48 |
| 18.... | do. | do. | do. | 289 | 224 | 0.77 |
| 19.... | do. | do. | do. | 387 | 273 | 0.71 |
| 20.... | do. | do. | do. | 387 | 275 | 0.71 |
| | | | | | Average | 0.68 |
| 21.... | Black gumbo...................... | Moist. | 50 to 66 | 53 | 41 | 0.77 |
| 22.... | do. | do. | do. | 81 | 70 | 0.86 |
| | | | | | Average | 0.82 |
| 23.... | Black gumbo...................... | Wet. | 50 to 66 | 50 | 25 | 0.50 |
| 24.... | do. | do. | do. | 79 | 70 | 0.89 |
| 25.... | do. | do. | do | 79 | 58 | 0.73 |
| | | | | | Average | 0.71 |
| 26.... | Black loam........................ | Moist. | 42 to 56 | 57 | 44 | 0.77 |
| 27.... | do. | do. | do. | 89 | 60 | 0.67 |
| 28.... | do. | do. | do. | 113 | 89 | 0.79 |
| | | | | | Average | 0.74 |
| 29.... | Black loam | Wet. | 42 to 56 | 57 | 38 | 0.67 |
| 30 .... | do. | do. | do. | 89 | 69 | 0.78 |
| | | | | | Average | 0.72 |
| 31.... | Yellow clay...................... | Moist. | 42 to 56 | 46.5 | 39 | 0.84 |
| 32.... | do. | do. | do. | 79.5 | 67 | 0.84 |
| 33.... | do. | do. | do. | 111.5 | 90 | 0.81 |
| | | | | | Average | 0.83 |
| 34.... | Yellow clay..................... | Very wet. | 42 to 56 | 79.5 | 42 | 0.53 |
| 35.... | do. | do. | do. | 111.5 | 52 | 0.47 |
| | | | | | Average | 0.50 |

FIG. 18.—TEST OF FRICTIONAL RESISTANCE.

FIG. 19.—DEEP DRAINS.

groups was about 15 per cent. The series of tests was not sufficiently extensive to warrant any conclusions as to variations in the coefficient of frictional resistance with variations in the load or in the velocity of the moving element.

The pull or force required to start the load in motion has not been shown in Table 2, for it is believed that, in order to make a reasonably close determination of the starting force, greater refinements are necessary than could be applied in the field experiments in question. It should be noted, however, that, in view of the method of testing used, the shearing resistance (due to cohesion) of the material under test was not brought into play.

The average of the results of all the experiments shows a coefficient of frictional resistance, including adhesion, equal to about 0.61. This figure, however, affords no reliable index, as is apparent from the wide variation in values corresponding to the different kinds of soil tested. The interpretation of the results as applying to design, therefore, is essentially a matter of judgment. The variations referred to are so great that the only safe course was to assume that the worst soil conditions (as regards frictional resistance) might exist under any bay. East of Buttress 36 (Plate II) the clayey content of the overburden was somewhat greater than west thereof, and hence it seemed reasonable to assume that there the soil is less subject to percolation and has a somewhat greater bearing value. Nevertheless, no modification was made in the matter of frictional resistance on this account.

The coefficient of frictional resistance for the clayey foundation soil (wet) was, for design purposes, finally assumed to be 0.33. The writer believes that for none of the soils at the dam site is the corresponding coefficient less than 0.25.

*Frictional Resistance of Shale.*—Here, again, there are but few data available; perhaps the most pertinent are those given by Mr. E. L. Lasier in his article on "Tests of Frictional Resistance of Concrete on Shale."* These tests were made under the direction of the writer at the site of the State Line Dam of the Hydro-Electric Company of West Virginia, on Cheat River, in West Virginia, and cover, not merely the frictional resistance of concrete on shale, but also that of shale on shale.

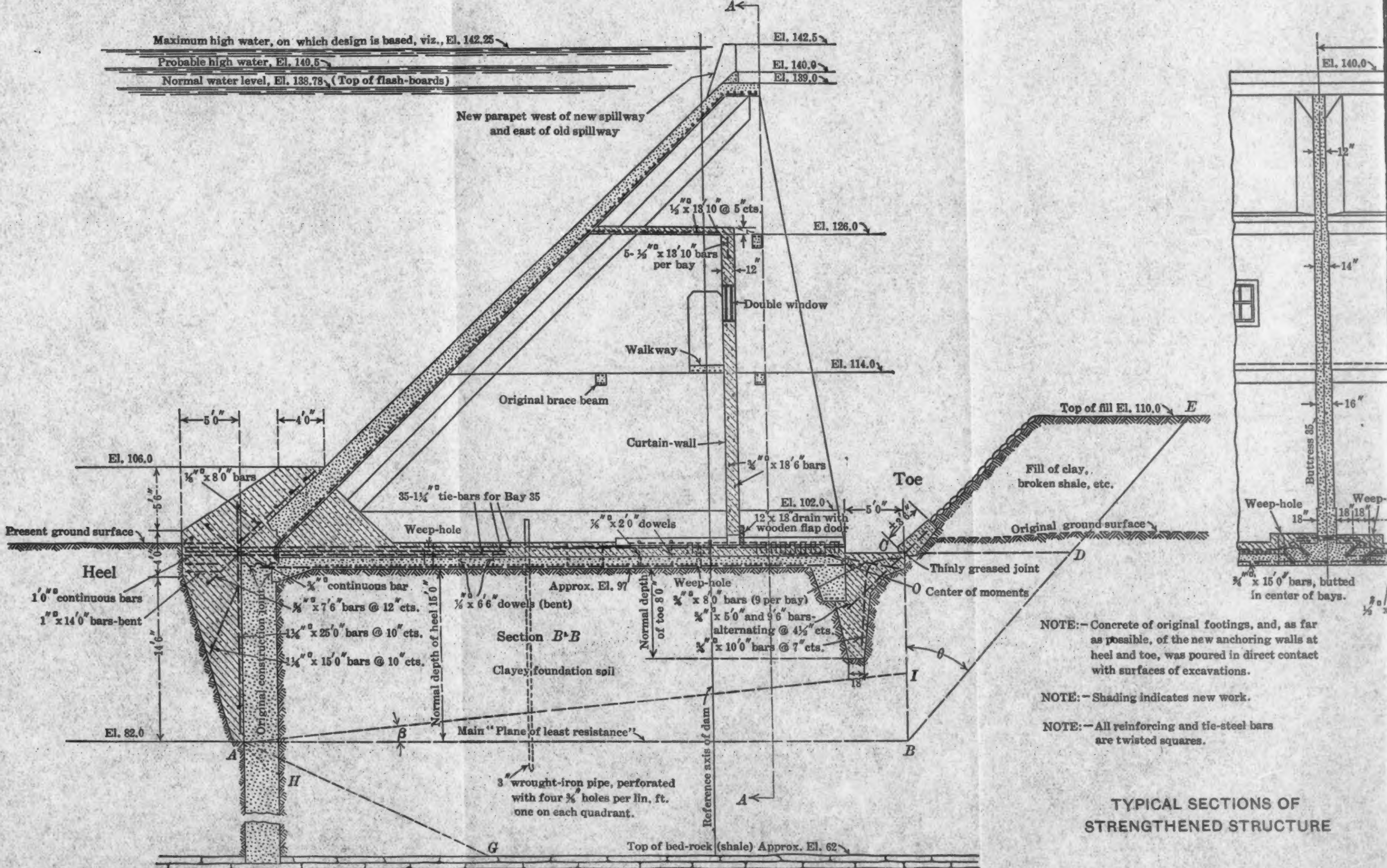
* *Engineering News*, July 22d, 1915.

At the Stony River dam site the shales have pronounced horizontal laminations.* Hence the results in Mr. Lasier's Table 1 apply, the tests summarized in that table having been made on quarried blocks of shale. There the coefficient of frictional resistance for the shale specimens moving on the stationary shale blocks averaged 0.61 when the surfaces in contact were dry, and 0.55 when they were wet. In view of the apparent differences in physical characteristics, it seemed proper to adopt assumptions even more conservative than those shown in Mr. Lasier's tests. This is especially true as regards those Stony River shales near the surface of bed-rock. Inasmuch as these had been exposed to disintegration ages ago, clay seams or laminations had been formed. The coefficients adopted are given in Table 1.

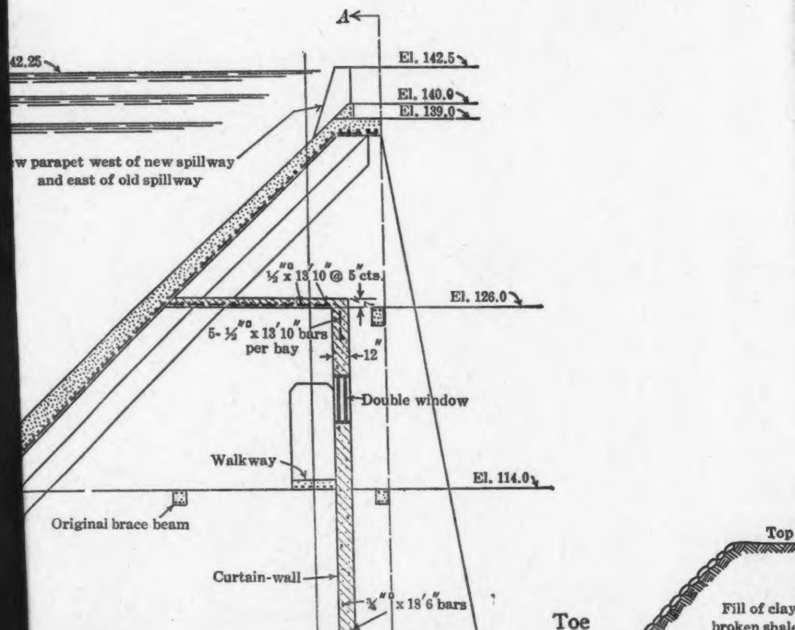
*Stability of Original Structure Against Sliding.*—The stability of a dam against failure of a given kind is ordinarily measured by its so-called factor of safety—which perhaps may be more correctly termed its "ratio of safety". This is the ratio of the forces resisting failure in a certain manner to the forces tending to cause such failure; but, whether or not a ratio of safety really means anything depends on the assumptions made in deducing that ratio; and, as previously suggested, it is seldom that two engineers working entirely independently arrive at exactly the same results as regards the ratio of safety. The writer makes it a practice to study the stability of a structure under several different sets of assumed conditions. Sometimes, too, after having ascertained the ratio of safety under the conditions to which it appears a dam will be subjected, it is instructive to see how severe, perhaps even how unreasonably severe, the conditions can be made without reducing the ratio of safety below unity.

In the present instance the stability of the dam was examined under the assumption of a "normal maximum load" and also under the "most severe conditions within the limits of reason", all, of course, as they appear to the writer. These several load assumptions are set out in Table 3, for a typical section of the dam, *viz.*, at Bay 35 (Plate V), and for the sections of maximum height, both on the basis of the original structure and also on the basis of the structure as strengthened during the reconstruction.

* Unlike the foundation shale at the Stony River site, the shales of the State Line site are not continuously laminated within themselves in any one plane. Hence the condition applying to the State Line site is more nearly that summarized in Table 2 of Mr. Lasier's article, which shows ratios of horizontal moving force to resistance to sliding (including resistance to shear in shale and concrete), the lowest of which is approximately 3.0.

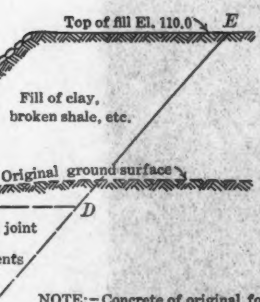


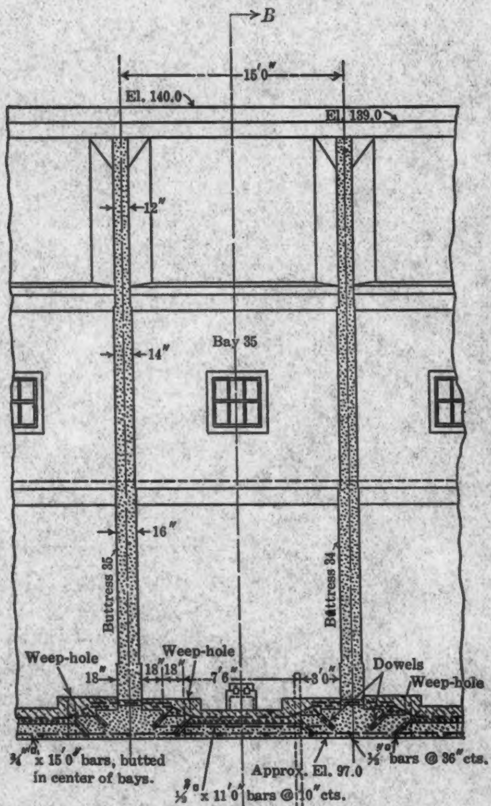

TYPICAL SECTIONS OF STRENGTHENED STRUCTURE



A
42.25
El. 142.5
El. 140.0
El. 139.0
w parapet west of new spillway and east of old spillway
½"⌀ x 13'10" @ 5" cts.
El. 126.0
5-½"⌀ x 13'10" bars per bay
12"
Double window
Walkway
El. 114.0
Original brace beam
Curtain-wall
¾"⌀ x 18'6" bars
Toe
Top
Fill of clay
broken shal
for Bay 35
El. 102.0
12 x 18 drain with wooden flap door
5'0"
⅜"⌀ x 2'0" dowels
Original gro
C
D
Thinly greased joint
O Center of moments
Approx. El. 97
Weep-hole
Normal depth of toe 8'0"
¾"⌀ x 8'0" bars (9 per bay)
¾" x 5'0" and 9'6" bars-alternating @ 4½" cts.
¾"⌀ x 10'0" bars @ 7" cts.
6" dowels (bent)
Section B-B
Clayey foundation soil
NOTE:
NOTE:
NOTE:
θ
I
18
"Plane of least resistance"
B
Reference axis of dam
3" wrought-iron pipe, perforated with four ⅜" holes per lin. ft. one on each quadrant.
A
G
Top of bed-rock (shale) Approx. El. 62

PLATE V.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
SCHEIDENHELM ON
RECONSTRUCTION OF
STONY RIVER DAM.




Section *A-A*

NOTE:—Concrete of original footings, and, as far as possible, of the new anchoring walls at heel and toe, was poured in direct contact with surfaces of excavations.

NOTE:—Shading indicates new work.

NOTE:—All reinforcing and tie-steel bars are twisted squares.

TYPICAL SECTIONS OF
STRENGTHENED STRUCTURE

TABLE 3.—LOCAL ASSUMPTIONS FOR ANALYSIS OF STABILITY OF STONY RIVER DAM.

| Factors. | ORIGINAL STRUCTURE. | | | | STRENGTHENED STRUCTURE. | | | |
|---|---|---|---|---|---|---|---|---|
| | Normal maximum load. | | Most severe conditions within limits of reason. | | Normal maximum load. | | Most severe conditions within limits of reason. | |
| | Sections of maximum height. | Typical section at Bay 35. | Sections of maximum height. | Typical section at Bay 35. | Sections of maximum height. | Typical section at Bay 35. | Sections of maximum height. | Typical section at Bay 35. |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
| (a) Elevation of head-water...... | 140.5 | 140.5 | 136.0 | 136.0 | 140.5 | 140.5 | 139.0 | 139.0 |
| (b) Ice pressure, in pounds per linear foot.................. | 0 | 0 | 20 000 | 20 000 | 0 | 0 | 20 000 | 20 000 |
| (c) Elevation of lower limit of equivalent full hydrostatic pressure against dam....... | 89.5 | 99.0 | 85.0 | 95.0 | 91.0 | 99.0 | 85.0 | 95.0 |
| (d) (1) Uplift pressure negligible. | Yes. | Yes. | No. | No. | Yes. | Yes. | No. | No |
| (2) Head, in feet, of equivalent uplift pressure (acting on one-half of affected area of base) at upstream edge of structure, decreasing uniformly to zero at downstream edge of footings, in case of original structure, and at deep drains in case of strengthened structure.............. | ...... | ...... | 48.0 | 39.0 | ...... | ...... | 54.0 | 44.0 *in re* sliding. 42.0 *in re* over-turning. |
| (3) Length of base, in feet, over which uplift pressure is exerted.......... | ...... | ...... | 61.0 | 51.5 | ...... | ...... | 40.0 | 30.0 |
| (e) Coefficient of frictional resistance.............. | 0.33* | 0.33* | 0.25* | 0.25* | 0.33 | 0.33 | 0.25 | 0.25 |

* Resistance (to horizontal movement) on part of the original unreinforced cut-off wall and the original toe probably inconsiderable and treated as forming part of the frictional resistance of the foundation soil at the base of the footings.

NOTE.—No tail-water pressure in any case.

Referring to Table 3 it should be noted that, in connection with the question of frictional resistance, no attempt has been made in the case of the original structure to determine the additional resistance of the original cut-off wall and toe. Inasmuch as the cut-off wall was unreinforced throughout, and was not tied into the body of the dam except at a few places, it is believed that the resistance of the cut-off

wall to horizontal movement was not a reliable factor, and that, even when present, it was inconsiderable in amount. Likewise, the resistance of the original was probably negligible, because the toe was relatively shallow.

The assumed conditions of Table 3, in so far as pressure is concerned, are entirely arbitrary. In view of the fact that, both at the sections of maximum height and at the typical section of Plate V, the strengthened structure has an adequate cut-off, it is certain, of course, that the uplift pressure actually does not decrease uniformly from heel to toe, as has been assumed. However, everything considered, and especially in the absence of definite knowledge regarding the presence and character of the uplift pressure, it appears reasonable to base stability analyses on assumed "equivalent" conditions of uplift pressure (see Table 3, factor (*d*)-2). Manifestly, uplift pressure could not be exerted over the entire area of the base without causing flotation, which is impossible.

I.—Under "Normal Maximum Load".—Applying now the assumed "normal maximum load" conditions of Column (2), Table 3, to the maximum height sections of the original structure, and stating all quantities for one bay 15 ft. wide, the results are as follows:

| | |
|---|---|
| Weight of concrete.................. | 1 044 900 lb. |
| Weight of water on deck............. | 1 194 500 lb. |
| Sum of loads...................... | 2 239 400 lb. |

Total resistance to sliding, 2 239 400 lb. $\times$ 0.33 = 746 500 lb., approximately.

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = \frac{746\ 500\ \text{lb.}}{1\ 218\ 200\ \text{lb.}} = 0.61.$$

Or, *vice versa,* in order that this ratio may become equal to unity, the coefficient of frictional resistance must equal

$$\frac{0.33}{0.61} = 0.54.$$

II.—Under "Most Severe Conditions Within Limits of Reason".—Referring to the assumed conditions of Column (4), Table 3, the spillway crest (Elevation 136) has been adopted as the highest elevation at which ice pressure could be present, because flash-boards could

hardly have been used with safety on the original spillway. Uplift pressure has been assumed as acting along the entire width of the base of the original structure (though over only 50% of the area), because, in severely cold weather, the weep-holes through the footings were frozen up solidly and hence were ineffective. With all quantities stated, as before, in terms of a 15-ft. bay, the results are as follows:

| | |
|---|---|
| Weight of concrete.................... | 1 044 900 lb. |
| Water load on deck.................... | 985 500 lb. |
| Total downward pressure............. | 2 030 400 lb. |
| Deduct uplift pressure................. | 675 000 lb. |
| Net load........................ | 1 355 400 lb. |

Total resistance to sliding = 1 355 400 lb. × 0.25 = 338 850 lb.

| | |
|---|---|
| Horizontal water pressure............ | 1 219 200 lb. |
| Ice pressure............................. | 300 000 lb. |
| Total forces tending to cause sliding... | 1 519 200 lb. |

Hence the ratio of safety is

$$\frac{338\,850 \text{ lb.}}{1\,519\,200 \text{ lb.}} = 0.223.$$

Or, in order that stability against sliding may become equal to unity, the coefficient of frictional resistance must equal 0.25 divided by 0.223, *viz.*, 1.12. Evidently, therefore, the Stony River Dam as originally constructed could not be considered as having a reasonable margin of safety against sliding.

It would be interesting, of course, to know what coefficient of gross resistance to sliding obtained prior to failure. In order to derive the minimum value of such coefficient, the following assumptions are made, conforming to the conditions which actually existed, *viz.*:

(*a*) Head-water level at elevation of spillway crest, *viz.*, at Elevation 136;

(*b*) No ice pressure;

(*c*) Horizontal water pressure equivalent to full hydrostatic pressure of 46.5 ft., *viz.*, down to Elevation 89.5;

(*d*) Uplift pressure negligible;

(*e*) No tail-water pressure;

(*f*) Any resistance to horizontal movement furnished by the unreinforced cut-off wall or the toe-wall is treated as forming part of the gross resistance to sliding on the part of the foundation soil.

Under these conditions, the ratio of the force tending to cause sliding to the load above the base of the footings, for a 15-ft. bay, was

$$\frac{1\ 013\ 600 \text{ lb.}}{2\ 030\ 400 \text{ lb.}} = 0.50.$$

In other words, the coefficient of gross resistance to sliding, under the foregoing conditions, was at least 0.50; for, necessarily, the ratio of safety was at least unity.

Probably the resistance to horizontal movement, as expressed by such ratio or coefficient, was due in part to cohesion or resistance to direct shear within the foundation soil itself. That is, it is likely that failure would occur, not at the relatively rough surface of contact between the footing concrete and the foundation soil, but along an approximately horizontal plane in the body of the soil just below the base of the footings. Such resistance to shear will be discussed further in connection with the expedient adopted to increase the resistance to sliding in that portion of the dam which was not disturbed by the failure.

It should be noted that in the 65 days prior to the failure, during which period the dam withstood the pressure of the full reservoir, relatively cold weather obtained, and the ground-water was practically at its lowest level. In other words, it is likely that the frictional resistance would have decreased as the foundation soil became more saturated by leakage and ground-water during the following spring. Whether the dam would have stood, or how near it was to failure by sliding, no man can tell.

*Means to Increase Resistance to Sliding.*—Various expedients were considered for increasing the margin of safety of the original structure against failure by sliding. The method finally selected involved the construction of anchoring walls at the heel and toe of the original structure, essentially as shown on Plate V, which shows typical sections of the dam, though not those of maximum height. In order to explain more clearly the reasons for adopting this method

of increasing resistance to sliding, the alternatives considered will be briefly described and their advantages and disadvantages stated.

1.—Referring first to the principal feature of the adopted means, the essential advantages of the anchoring wall at the heel (the action and construction of which are discussed in more detail later herein) are as follows (Plate V):

(a) It utilizes the foundation soil under the dam, and in effect also a considerable quantity of the foundation soil immediately down stream from the dam, to increase the total weight which must be moved down stream in case the dam is to fail by sliding.

(b) The construction of such an anchoring wall at the heel seals the construction joint at the top of the original cut-off wall, this joint having in some places opened up sufficiently to allow appreciable, if indeed not serious, leakage.

(c) With such an anchoring wall, any down-stream movement on the part of the dam will tend to compact the underlying foundation soil, and, therefore, to make it less pervious.

(d) This was found to be the most economical method of obtaining the desired margin of safety against failure by sliding.

The only disadvantage of constructing an anchoring wall at the heel alone lay in the resulting high unit pressure against the clay immediately down stream from such "heel"—as the new anchoring wall at the heel of the dam has for convenience been styled.

2.—With especial reference to that portion of the original structure which remained intact, there was, of course, no opportunity to flatten the slope of the deck, and thus obtain a greater water load.

3.—The loading of the footings of the hollow structure, by using concrete, rock, or earth, was considered, and was rejected because:

(a) Such additional material would increase the load on the already too heavily loaded foundation soil;

(b) The original footings would have required even more strengthening than was considered necessary with the means finally adopted;

(c) Such loading would have required special provision for inspection of the drainage system; and

(d) The expedient was limited in extent, and, taken by itself, was not considered to be sufficient.

4.—Various structural additions at the toe of the dam were considered; among them an L-shaped anchoring wall such as that shown on Plate V. This method of obtaining additional resistance to sliding was not open to the objections to the proposed loading of the original footings. However, if adopted to furnish the entire required increase in resistance to sliding, it would have been subject to certain different objections, *viz.*:

(*a*) For the same depth below original ground surface, generally speaking, less increase in resistance to sliding results than in the case of an anchoring wall at the heel alone; for the toe-wall does not utilize the foundation material under the body of the dam.

(*b*) In the case of this particular clayey soil, the toe-wall, like the "heel", is limited in efficacy by the safe bearing value of the soil in compression.

(*c*) In case of any yielding at the toe of the dam, either downward or down stream, the effect might be to cause either cracking in the original cut-off wall or, at the very least, the opening of the construction joint at the top of the old cut-off (Plate V). Thus, water under reservoir pressure might immediately be admitted under the base of the dam.

On the other hand, the anchoring wall at the toe has certain advantages not possessed by the "heel", *viz.*:

(*d*) By means of its vertical member, the toe-wall confines the foundation soil under the original footings of the dam, thus increasing the safe load to which that soil may be subjected.

(*e*) The horizontal member of the toe-wall in effect increases the width of the base of the dam, thus decreasing the unit vertical load on the foundation soil, as will be shown later in more detail under the heading "Footings".

It is clear, however, that in cases where it is necessary similarly to increase the margin of safety of a dam against failure by sliding, but where there is no opportunity to construct an anchoring wall at the heel, an anchoring wall at the toe can be made to answer the purpose. Generally speaking, it is merely a matter of proportioning such a toe-wall properly.

5.—Investigation was made of the desirability of constructing reinforced concrete columns at the toe of the dam, inclining downward and approximately parallel to the line of action of the resultant load on the dam. This proposal was rejected, partly because the result would be equivalent to placing the dam on relatively unyielding supports—stilts, as it were—thus changing the character of the stresses in the original footings.

6.—The construction of a heavy block of concrete, approximately square in cross-section and extending along the entire length of the toe of the dam, was considered and rejected because the L-shaped toe-wall transmits vertical and horizontal loads into the foundation material to the same extent, but with much less concrete and excavation.

In view of the foregoing considerations, it was concluded to use anchoring walls at both heel and toe; for, by means of the toe-wall, it was possible to reduce to a safe value the otherwise excessive horizontal load on the foundation soil at the heel. The reconstructed dam acts essentially as a monolith; hence down-stream deflection or yielding must be equal at heel and toe. Inasmuch, then, as the load on the foundation soil is proportional to the deflection, the unit horizontal loads at heel and toe are approximately equal.

Likewise, at the new spillway of the dam, an anchoring wall at the heel was used. Here advantage was taken of the opportunity of combining the cut-off and the anchoring device in a single wall, as shown on Plate III. As the foundation of the new spillway consists of shale, the problem of keeping within the safe load against the material at the heel was not difficult, and hence no achoring wall was required at the toe.

*Analysis of Anchoring Walls.*—The principle on which the design of the anchoring walls is based applies both in the case of the strengthening of the original structure and in that of the construction of the new spillway. The detailed explanation, however, will be made with reference to the typical bulkhead section at Bay 35, as shown on Plate V.

The use of such anchoring walls, to furnish specific and reliable resistance to sliding, by utilizing the weight of the foundation material underlying, and immediately down stream from, a dam, is believed to be new. It was first applied by the writer in the case of the State Line Dam of the Hydro-Electric Company of West Virginia, on Cheat River in West Virginia.

Referring now to Plate V, the anchoring wall at the heel is tied into the dam by twisted, square, steel bars extending through openings broken through the base of the original deck of the dam. The heel itself consists essentially of two cantilever members of reinforced concrete, extending, respectively, above and below the steel which ties the heel to the original structure. A relatively long cantilever member extends downward immediately adjacent to the up-stream face of the original cut-off, and a relatively short cantilever member extends upward immediately adjacent to the base of the original deck. Both cantilevers are suitably reinforced against stresses due to bending moment and shear.

Although some of the horizontal water pressure is exerted directly against the anchoring wall or heel, the greater portion is exerted against the deck, and by the deck is transmitted into the buttresses. The tie-steel previously referred to is embedded in the reinforced concrete footing strengthening, which in turn is thoroughly tied into the base of each adjacent buttress, as shown on Plate V. Thus the tie-steel receives from the buttresses and the footings that portion of the horizontal water pressure which is not exerted directly against the heel, or has not been taken up by the frictional resistance of the foundation soil immediately in contact with the footings.

The lower cantilever member of the anchoring wall in turn transmits such surplus load into the foundation soil down stream from the heel. The fact that the stress is transmitted through the medium of the original cut-off wall is immaterial, except to the extent that the cut-off wall serves to transmit some of the load into the foundation soil below, and down stream from, the bottom of the heel. The reaction at the top of the lower cantilever, of course, is taken up by the upper and shorter cantilever and transmitted through the deck, in part directly into the buttresses and in part into the approximately triangular fillet of new concrete placed in the angle between the deck and the footings (Plate V). Ordinarily, this fillet acts as an arch to transmit the reaction back into the buttresses and footings. In certain cases, however, where steps occur in the footings at the centers of bays, it was necessary to reinforce the triangular fillets to act as cantilever beams supported at the buttresses and extending outward to the centers of the adjacent bays.

The effect of the anchoring wall at the heel is essentially to lower the main "plane of least resistance" to sliding from its original posi-

tion, at or immediately under the base of the footings of the dam, to a position approximately at the elevation of the bottom of the new anchoring wall at the heel. By "plane of least resistance" to sliding is meant the surface or surfaces, approximating a plane or planes extending through the foundation material, along which the total resistance to horizontal movement (sliding) is less than along any other surfaces or approximate planes.

In so far as resistance to sliding is concerned, the result of such lowering of the main "plane of least resistance" is to cause the weight of the foundation material in advance of the anchoring wall to be utilized at least as effectively as the weight of the body of the dam itself. This utilization of the weight of the foundation material occurs in two ways, as will be apparent by reference to Plate V. Let it be assumed that, for the conditions existing in the typical section there illustrated, the "planes of least resistance" are represented by the lines, *A-B* and *B-E*, respectively. Then

1.—The foundation material lying above *A-B* and to the left of *B-C* must, in case of failure, slide along the plane, *A-B*. This is also true of the concrete structure (including the anchoring wall) and the superimposed water load. The resistance to sliding of these several elements is represented by the product of their weight multiplied by the coefficient of frictional resistance.

2.—The foundation material lying between *B-C* and *B-E* must, in case of the failure of the dam by sliding, move simultaneously down stream and upward along the plane, *B-E*. Such movement of this material, therefore, is opposed, not merely by frictional resistance, but also by the force of gravity. The resistance of this particular portion of the foundation material is analyzed subsequently in more detail.

The location of the "planes of least resistance" is dependent on the conditions existing in any particular case. For the conditions shown on Plate V, a general statement may be made, as follows:

It is evident that, assuming proper design and construction of the anchoring wall at the heel of the dam, the main "plane of least resistance" cannot begin above *A*. Neglecting for the moment the existence of the original cut-off, it is further evident that the main "plane of

least resistance" could not extend downward in a direction such as *A-G*. In other words, the resistance along any plane dipping below the horizontal is necessarily greater than the resistance along the plane, *A-B*. Whether the "plane of least resistance" extends horizontally or in an upward direction from *A* is a matter for determination by trial computation. In this particular case, of course, the old cut-off wall is a factor, even though its effect may be indefinite. For instance, were the break in it to be in an upward direction, such as *A-F*, its effect on the location of the main "plane of least resistance" would be nil, or at least negligible. On the other hand, a break in a downward direction, such as *A-H*, might, owing to the strength of the concrete cantilevering below the elevation of *A*, cause the initial point of the main "plane of least resistance" to be at a lower elevation, for example, at *H* instead of *A*.

To the left of the vertical plane represented by the line, *B-C*, passing through the down-stream edge of the new toe-wall of the dam, the foundation material is confined by the weight of the structure shifted, as it were, toward the down-stream side of the footings by the horizontal thrust of the water load. In effect, this weight is greatest per square foot at the very down-stream edge of the toe-wall, which is practically monolithic with the original structure. The line or plane of failure could not turn upward (from the main "plane of least resistance") on the left side of *B-C* unless the foundation material underlying the dam, and above such new and increased slope, were to fail in compression, or were to "flow".

Moreover, at the toe there is a concentration or transfer of horizontal thrust into the foundation material. In the present instance this transfer of thrust is accomplished partly through frictional resistance between the soil and the toe-wall (especially the horizontal leg thereof), and partly through direct bearing of the toe-wall (especially the vertical leg thereof) against the soil.

It is probable, therefore, that the main "plane of resistance" would end approximately below the down-stream edge of the toe-wall, as at *B*.

If, then, the main "plane of least resistance" is represented by *A-B*, it follows that unless (1) the foundation material fails in compression, as along *B-C*, or (2) the coefficient of frictional resistance is too great, the "plane of least resistance" will turn upward at *B*. Generally speaking, the slope of this plane must be determined by trial,

although, under certain conditions, the slope may be determined by direct computation.

*Theory of Resistance at Toe.*—The resistance of the foundation material at the toe of the dam, by which, on Plate V, for instance, is meant the material to the right of *B-C,* is a factor sufficiently important to warrant more detailed explanation. Practically, on account of the character of the material in question, the results of theoretical analysis represent the actual conditions only approximately. The refinement of such analysis, however, is worth while, because it aids in obtaining a thorough understanding of the case.

The problem is essentially one of "passive thrust". The analysis adopted as applying to the general case is based on the assumptions that:

(*a*) The foundation material, *viz.,* earth, is homogeneous and granular;

(*b*) The material is without cohesion;

(*c*) The coefficient of frictional resistance (being the same as the coefficient of internal friction) $= f$;

(*d*) In case of sliding, failure of the foundation material at the toe will occur along a plane, such as *B-E*;

(*e*) The resistance of the foundation material to compression (as in the vertical plane, *B-C*) is so great as not to be the limiting factor in toe resistance;

(*f*) The load on the foundation material in a vertical plane (such as *B-C*) increases in intensity regularly from zero at *C* to a maximum at *B*;

(*g*) It is necessary to the stability of a granular mass that the direction of the pressure of the portions into which it is divided by any plane should not at any point make with the normal to that plane an angle exceeding the angle of repose.*

Under the foregoing assumptions, it would appear that, in case of failure of the dam by sliding, a wedge-like section of the foundation material would be forced upward approximately between and along *B-C* and *B-E*. The toe resistance—in other words the resistance to movement on the part of such wedge-like section—depends on whether there is frictional resistance along the vertical plane, *B-C,* such as

* Rankine's "A Manual of Applied Mechanics", 17th Ed., p. 213, Theorem I.

there is along the inclined plane, $B$-$E$. There appears to be no valid objection to the assumption that the resistance along the vertical plane is equal to that along the inclined plane. However, the results will be stated on each basis, the more conservative assumption being that there is no frictional resistance in the vertical plane, $B$-$C$:

*Condition I.*—Without friction in the vertical plane, $B$-$C$. (See Fig. 20.)

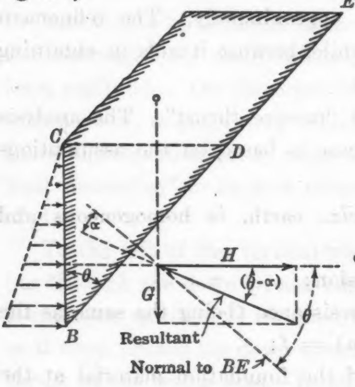



FIG. 20.

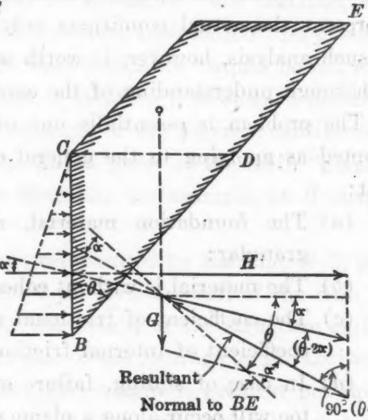



FIG. 21.

If $H$ = horizontal pressure in the plane, $B$-$C$, against the wedge of earth, $C$-$B$-$E$, sufficient to cause movement of the wedge;

$G$ = weight of material (per unit length of dam) included between $B$-$C$ and $B$-$E$;

$\alpha$ = angle of friction (*viz.*, the angle the tangent of which is $f$);

$\theta$ = angle between "plane of least resistance", $B$-$E$, and the vertical (plane $B$-$C$).

Then $H = G \cot. (\theta - \alpha)$, since the friction, along the plane, $B$-$E$, resists both the thrust, $H$, and movement of the weight, $G$.

For the purpose in question, it is necessary to determine the minimum value of $H$. In the general case, with an irregular surface boundary between $C$ and $E$, it is necessary to determine such a minimum value by trial. Ordinarily, this can most easily be accomplished by plotting a curve with values of $\theta$ as abscissas, and corresponding computed values of $H$ as ordinates. The lowest value of $H$ shown by such a curve is, of course, the desired value.

However, in the special case in which the upper bounding surface is horizontal—as represented on Plate V by *C-D*—the minimum value of $H$ may be stated mathematically by expressing $G$ in terms of $\theta$ and then placing the first differential of the resulting equation equal to zero. Accordingly, it is found that $H$ is a minimum when

$$\theta = 45^\circ + \frac{\alpha}{2}.$$

Evidently, in this special case the critical value of $\theta$ (giving the minimum value of $H$) cannot be less than 45 degrees.

*Condition II.*—The frictional resistance in the vertical plane, *B-C*, equals $f$. (See Fig. 21.)

In the general case, under this condition,

$$H = G \cos. \alpha \frac{\cos. (\theta - \alpha)}{\sin. (\theta - 2\alpha)}$$

It follows that, since $H$ becomes infinitely great when $\sin. (\theta - 2\alpha) = 0$, $\theta$ must be greater than $2\alpha$. Again, under ordinary physical conditions, $G$ and, hence also, $H$, become infinitely great for $\theta = 90$ degrees. Therefore, excepting for failure in compression, failure cannot occur unless $\alpha$ is less than 45°—in other words, unless $f$, the tangent of $\alpha$, is less than 1.0.

In the special case where the upper surface of the toe material lies in the plane, *C-D*, the minimum value of $H$ may be determined mathematically, as under Condition I, it being found that $H$ is a minimum when

$$\theta = \tan.^{-1} \left( \frac{2 \tan. \alpha + \sec. \alpha \sqrt{2}}{1 - \tan.^2 \alpha} \right)$$

*Actual Toe Resistance.*—Under many, if indeed not under most, foundation conditions, the toe resistance is limited practically, not by the theoretical minimum values of $H$ as expressed in the foregoing paragraphs, but by a third condition, namely, the bearing value of the foundation material. This is the case at the Stony River site. For design purposes, the maximum load in the plane, *B-C*, under the conditions illustrated on Plate V, was limited to the safe bearing value, in lateral compression, of the soil at the site. This value was assumed as 4 000 lb. per sq. ft. In other words, the average resistance to sliding in the vertical plane, *B-C*, was limited to 2 000 lb. per sq. ft.

To illustrate the possibilities of toe resistance, as well as the probable actual values, reference is again made to the typical section of the strengthened structure at Bay 35, as shown on Plate V, all calculations being based on normal maximum load conditions, with $f$ assumed $= 0.33$:

Assuming first that the main "plane of least resistance" to sliding is horizontal (see *A-B*, Plate V), and assuming that there is no frictional resistance in the vertical plane, *B-C*, the inclined "plane of least resistance" at the toe is found to make an angle, $\theta$, of 44° with the vertical for the minimum value of $H$ (toe resistance to sliding). The corresponding total resistance to sliding is approximately 140 080 lb. per lin. ft. If, instead, the frictional resistance in the vertical plane, *B-C*, is assumed to be equal to that in the inclined "plane of least resistance", the latter is found to make an angle of 67° with the vertical, with a corresponding total resistance of about 183 530 lb. per lin. ft. However, if limited by the assumed safe bearing value of the soil in lateral compression, that is 4 000 lb. per sq. ft., the total resistance to sliding is only 115 080 lb. per lin. ft.

Assuming now at random a different location of the main "plane of least resistance", *viz.*, *A-I* (Plate V), making an angle, $\beta$, of 6° with the horizontal, it is found that, without frictional resistance in the vertical plane, *I-C*, the angle, $\theta$, is 39°, and the total resistance to sliding is about 131 200 lb. per lin. ft.; that with full frictional resistance ($f = 0.33$) in the plane, *I-C*, the angle, $\theta$, is 65°, and the corresponding total resistance is 159 300 lb. per lin. ft.; but that, if limited by the safe bearing value of the soil, the total resistance is only 121 650 lb. per lin. ft. In order to determine the minimum values of the total resistance to sliding for each of the three conditions just mentioned, enough different locations of the main "plane of least resistance" were assumed, and corresponding calculations were made, to warrant the plotting of a diagram in which the values of $\beta$ (the angle which the main "plane of least resistance" makes with the horizontal) and of total resistance to sliding, respectively, were plotted as rectangular co-ordinates.

It was evident from inspection of the results that, with the resistance at the toe limited by the safe bearing value of the soil, the minimum total resistance to sliding is afforded when the main "plane of least resistance" is in the horizontal position, *A-B*. However, from

the diagram which was based on the assumption that there is no frictional resistance in the vertical plane, $\beta$ was found to be approximately 8° for the minimum value of total resistance to sliding, equaling about 128 500 lb. per lin. ft. Correspondingly, a value of 8° 15′ was found for $\beta$ under the assumption that the coefficient of frictional resistance in the vertical plane too is 0.33, the minimum total resistance to sliding being approximately 152 700 lb. per lin. ft. Both these values of total resistance to sliding are greater than that obtained when the resistance at the toe is limited by the safe bearing value of the soil, *viz.*, 115 080 lb. per lin. ft. Consequently, for design purposes, the "planes of least resistance" under "normal maximum load" for the section shown on Plate V were assumed to be represented by *A-B* and *B-C*.

It is interesting to note to what extent the total resistance to sliding at Bay 35 was increased in the reconstruction. Retaining still the "normal maximum load" conditions, and with resistance at the toe limited by the safe bearing value of the soil, the total resistance to sliding was increased from approximately 41 310 lb. per lin. ft., in the case of the original structure, to approximately 115 080 lb. per lin. ft., in the case of the strengthened structure. In other words, the total resistance to sliding was increased by about 178 per cent. Correspondingly, on the basis of the assumption that the resistance at the toe is not limited by the safe bearing value of the soil, but that no frictional resistance exists in the vertical plane at the toe, the total resistance to sliding would be increased by about 211%; and if, in addition, the coefficient of frictional resistance in the vertical plane is assumed to be 0.33, the increase would have been about 270 per cent.

Of course, there is undoubtedly a considerable difference between theoretical and actual conditions. Thus, referring to the foregoing assumptions on which the theoretical analysis has been based, earth is rarely, if ever, strictly homogeneous. The coefficient of frictional resistance varies, and the "planes of least resistance" are probably never true planes. Crude experiments made by the writer indicate that, even in the case of such granular material as sand, the surface along which failure occurs is by no means a true plane. Under the Stony River conditions, the "planes of least resistance" are affected also by boulders. Again, any cohesion existing within the founda-

tion soil causes the resistance to sliding to be greater than according to the preceding analysis. It is evident, however, that procedure along the lines indicated is amply conservative and safe.

It may not be amiss to point out that anchoring walls such as those described may be applied properly and economically in the case of rock foundations, particularly when the foundations consist of laminated rock with relatively low frictional resistance. By this means a considerable quantity of the foundation rock can be made to afford resistance to sliding just as reliably as though the corresponding weight consisted of concrete in the body of the dam. Generally speaking, anchoring walls, even if not used for the purpose of reducing the quantity of material in the body or superstructure of a dam, will, at least, increase the margin of safety.

*Stability of Strengthened Structure Against Sliding.*—The design of the features entering into the strengthening of those portions of the dam which remained intact conformed as closely as practicable to the general criterion that the ratio of the probable forces resisting sliding should be twice the probable forces tending to cause sliding.

It will be noted from Plate V that an earth fill has been placed down stream from the typical section there shown. Fig. 22 shows more clearly the limits and height of this fill, which extends along the entire length of the higher sections of the dam, *viz.,* between the old and the new spillways. Referring again to Plate V, it is evident that the presence of this earth fill increases the quantity of material lying with the angle, $\theta$. Moreover, the greater the coefficient of frictional resistance of the foundation soil, the greater the angle, $\theta$, and hence the greater the effective resisting weight due to the earth fill. The material for the fill was obtained largely from the excavation for the new spillway channel.

As in the case of the original structure, the margin of safety resulting from the work of strengthening will be illustrated by several examples under different sets of conditions. Thus, for the strengthened sections of maximum height, the results are as follows:

I.—Under the assumed "normal maximum load" conditions, as set out in Column (6), Table 3,

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = 2.03.$$

Fig. 22.—Stony River Dam as Reconstructed. View from West Bank.

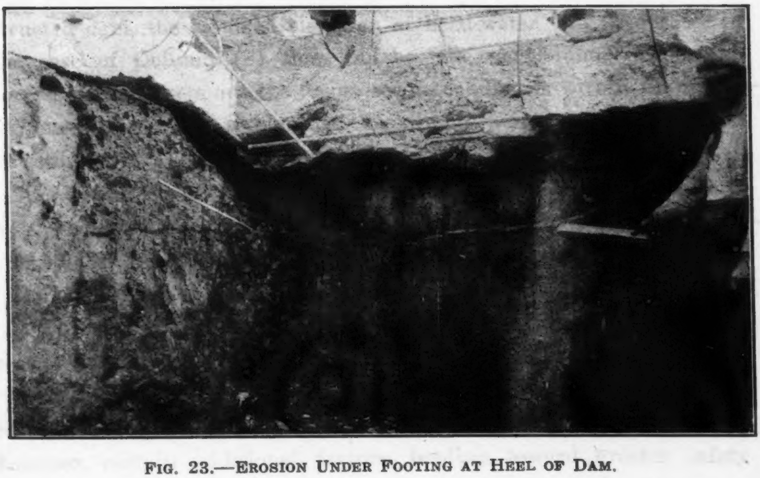


Fig. 23.—Erosion Under Footing at Heel of Dam.

For the original—that is, the unstrengthened—structure, the corresponding ratio, as previously explained, was 0.61. By comparison of the conditions set out in Columns (2) and (6), Table 3, it will be noted that the only difference in the assumed conditions is that in the case of the strengthened structure, Column (6), factor (*c*), the assumed elevation of the lower limit of equivalent full hydrostatic pressure against the dam was raised somewhat. This was warranted by the fact that considerable pervious material was taken out of the river-bed, immediately up stream from the dam, and replaced by a back-fill of impervious clay, the back-fill in turn being covered in part with a reinforced concrete protecting mat, especially near the sluice-gates (see Plate IV).

II.—Under the "most severe conditions within the limits of reason", as applied to the strengthened structure (see Column (8), Table 3),

$$\frac{\text{Forces resisting sliding}}{\text{Forces tending to cause sliding}} = 1.07.$$

This ratio is to be compared with the ratio of 0.22 for the sections of maximum height, of the structure as it stood originally. The comparison is the more striking when it is noted that, inasmuch as 3-ft. flash-boards can be used safely on both spillways of the reconstructed dam, the assumed elevation of head-water is 3 ft. greater in the case of Column (8) than in the case of Column (4), which correspondingly sets out the "most severe conditions within the limits of reason" for the sections of maximum height of the original structure.

As to the typical strengthened section at Bay 35, shown on Plate V, the margin of safety against sliding is represented as follows:

III.—Under the "normal maximum load" conditions set out in Column (7), Table 3, the ratio of safety is 2.14.

IV.—Under the "most severe conditions within the limits of reason", as set out in Column (9), Table 3, the ratio of safety is 1.12.

These ratios of course, would be bettered by any increase in the respective coefficients of frictional resistance assumed. There are, however, certain additional factors, tending toward greater safety against sliding, which apply in the case of the strengthened portions

of the dam founded on the clayey over-burden, but in none of which was any specific reliance placed, namely:

(*a*) In the structure as strengthened there is no tendency to break at the top of the cut-off wall. The resistance of the old cut-off wall to shear or rupture at the bottom of the new "heel" is, therefore, a considerable factor in resisting sliding.

(*b*) Again, if such shear or rupture in the cut-off wall should occur below the bottom of the "heel", or should extend below the bottom of the "heel" in a direction such as *A-H*, Plate V, the effect would be to lower still further the "planes of least resistance" and, consequently, to increase the resistance to sliding.

(*c*) The existence of cohesion or resistance to shear in the clayey foundation soil is by no means speculative even though the amount of such resistance may not be readily and reliably determinable.

*Tests of Shearing Value of Clay.*—The writer has found no previously published data on this subject, and therefore believes that certain tests made during the Stony River reconstruction by Mr. D. N. Showalter, Resident Engineer, will be of general interest. The apparatus contrived by Mr. Showalter for making the test was simple and yet quite effective.

Specimens of clay were dug out of the foundation over-burden and cut to such shape as to fit the apparatus. The shearing was then accomplished by slicing off the clay, as it were, with a 2 by 6-in. piece of wood which was forced across the specimen by a small set of blocks and tackle. The tension in the rope was measured by a spring balance. The specimens tested were fairly moist. The results of these tests of the shearing values of several different clayey soils are given in Table 4.

From inspection of the data in Table 4, it will be noted that, as was to be expected, the presence of sand in the clay very materially lessens the shearing value. However, even among Specimens 1 to 8, inclusive, there is considerable variation among the results. The effect of the length of time of application of a given shearing load was not investigated, though time is undoubtedly a very important

factor. The results clearly indicate that here is a fruitful field for further experimentation.

TABLE 4.—RESULTS OF TESTS OF SHEARING VALUE OF FOUNDATION SOILS AT STONY RIVER DAM SITE.

| Experiment No. | Character of soil under test. | Initial area under shearing stress, in square inches. | Load* at failure, in pounds. | Shearing value, in pounds per square inch of initial areas. |
|---|---|---|---|---|
| 1 | White clay. | 9.0 | 109 | 12.1 |
| 2 | " " | 4.8 | 128 | 26.6 |
| 3 | " " | 6.1 | 126 | 20.7 |
| 4 | " " | 8.7 | 226 | 26.0 |
| 5 | " " | 7.7 | 176 | 22.8 |
| 6 | Black gumbo. | 19.2 | 184 | 9.6 |
| 7 | " " | 19.2 | 309 | 16.1 |
| 8 | Black loam. | 13.7 | 219 | 16.0 |
| 9 | Sandy yellow clay. | 19.2 | 89 | 4.6 |
| 10 | " " " | 14.6 | 39 | 2.7 |
| | | | Average...... | 15.7 |

*Including proper allowance for weight of apparatus.

NOTE.—All soil was in natural state of moistness.

To the writer, it appears that it is hardly proper, in the light of present information, that the shearing value of clay should be taken into account over and above the resistance to sliding developed under such conditions as obtained in the tests of frictional resistance reported in Table 2. The resistance of the clay to shear, however, affords an added margin of safety, indefinite though the extent of the added margin may be. Accordingly, it is interesting to consider what might be the effect of such resistance to shear in a typical case, for example, the section at Bay 35 under "normal maximum load" conditions.

Referring to Plate V, it will be remembered that, in case of failure of the dam by sliding, it was assumed that failure along *A-B* would occur by sliding of clay on clay, and that failure along *B-C* would occur by the crushing of the clay or other foundation material. Hence it is only along *A-B* that the resistance of the clay to shear could increase the total resistance to sliding. Such resistance to shear, however, would act instead of, and not in addition to, the frictional resistance along *A-B*, which, under the assumed conditions, is about 75 000 lb. per lin. ft. of dam. The average shearing value shown by the tests reported in Table 4 is 15.7 lb. per sq. in. If, then, a resistance to shear of 15 lb. per sq. in., for instance, existed in the

53.5 lin. ft. of clay along *A-B*, the total resistance along *A-B* (exclusive of resistance to rupture on the part of the old cut-off wall) would be about 115 000 lb. per lin. ft. of dam. Thus the total resistance to sliding would be increased by about 40 000 lb. per lin. ft.

However, for the reasons stated previously, and because of the varying characteristics of the foundation soil, resistance of the clayey soil to shear was not relied on to furnish resistance to sliding.

*Resistance to Sliding at New Spillway.*—Turning now to the new spillway section, as shown on Plate III, it will be noted that, by reason of the use of the combined anchoring wall and cut-off at the heel of the structure, the principal "plane of least resistance" (at the elevation of the bottom of the anchoring wall) lies in relatively hard shale. Free from the hampering conditions imposed by a structure already existing on the site, it was possible at this particular place to secure more readily and economically given ratios of safety than obtain in those portions of the dam which were left intact after the failure. Thus the new spillway section has generally a greater margin of safety.

I.—Referring to the 15-ft. side section centering on Buttress 17 (Plate II), the ratio of safety against sliding under the "normal maximum load conditions" is 3.69. The conditions assumed in arriving at this ratio were essentially like those of Column (6), Table 3, except that, in accordance with Table 1, the coefficient of frictional resistance was assumed to be 0.5.

II.—For the same location in the new spillway, but under the "most severe conditions within the limits of reason," the ratio of safety against sliding is 2.03. In this case the assumed conditions were essentially like those of Column (8), Table 3, except that

(*a*) Head-water was assumed at Elevation 142.25, with no ice pressure; and

(*b*) The minimum coefficient of frictional resistance, in accordance with Table 1, was assumed as 0.40.

It is pertinent at this point to note, by reference to Plate V, that apparently no harm would result from uplift pressure exerted along the planes of contact between the footings of the strengthened structure and the foundation soil. That is, there would be practically no change in the vertical load above the probable "plane of least resist-

ance", such as *A-B*. Likewise, referring now to Plate III, it would appear that uplift pressure could exist in the laminations of the foundation shale rock immediately under the buttress footings (but above the elevation of the bottom of the anchoring wall) without affecting materially the stability of the new spillway section as regards sliding. Such uplift, however, would have a material effect on stability, as regards overturning.

## Resistance to Overturning.

In the case of a hollow dam with a deck slope of approximately 45° from the vertical, it is hardly necessary to inquire deeply into the question of stability as regards overturning. This will be apparent from the following illustrations:

I.—In the case of the typical section at Bay 35, in its original, unstrengthened condition (see unshaded portion of Section *B-B,* Plate V), the ratio of safety against overturning under the assumed "normal maximum load" conditions of Column (3), Table 3, was 3.65.

II.—For the same section under the "most severe conditions within the limits of reason", as set out in Column (5), Table 3, the corresponding ratio of safety was 1.14.

In deriving these ratios, moments were taken about the point, *O,* at the toe of the original structure, as indicated in Section *B-B* of Plate V, and it was assumed that, should the dam begin to overturn, the original cut-off wall would fail at the construction joint at the top of the wall. Furthermore, the horizontal thrust was considered to be taken up entirely by frictional resistance along the base of the footings; that is, no resistance to horizontal thrust was attributed to the body of the cut-off wall. In the second of the foregoing sets of assumptions cognizance was taken of the fact that, in severely cold weather, the weep-holes in the footings of the original structure freeze up solidly and become ineffective. Consequently, uplift pressure was considered as affecting a portion, assumed to be 50%, of the total area of the base.

Referring now to the typical strengthened structure at Bay 35, as shown on Plate V:

**III.—The ratio of safety against overturning under the assumed "normal maximum load" conditions of Column (7), Table 3, is 4.57.**

**IV.—Under the "most severe conditions within the limits of reason", Column (9), Table 3, the corresponding ratio becomes 1.68.**

**In developing the last two ratios it was assumed, as before, that in case of actual overturning, the original cut-off wall would fail at the construction joint at the top of the wall. However, as regards the effect of frictional resistance along the base of the main footings of the dam, in taking up horizontal thrust, such resistance was assumed to be equal to only 0.2 of the total concrete and water load, the remainder of the horizontal thrust causing a uniformly distributed load on the anchoring wall at the heel. The anchoring wall at the toe was assumed to carry no load. Such assumed distribution of the resistance to the horizontal thrust is obviously arbitrary, but affords the most unfavorable conditions as regards stability against overturning. The resultant of the assumed load on the "heel" would necessarily act at half the depth of the "heel".**

**Needless to say, the increased ratios of safety against overturning in the case of the strengthened structure were not sought after *per se*. They resulted from the provision made to increase the margin of safety against sliding.**

### FOOTINGS.

***Bearing Value of Foundation Soil.*—In the case of clayey soil, compressive loading causes appreciable yielding; hence the safe loading or bearing value is measured by the maximum allowable yielding. As the result of observations made at the dam site, under various conditions during the investigation period, and also of various tests of the soil at the site, it was concluded that the yielding due to a compressive loading of about 5 000 lb. per sq. ft. would not exceed 0.1 in. provided the soil is confined against "flow".**

**It was considered preferable not to exceed a maximum loading of 4 000 lb. per sq. ft., but somewhat greater loads were allowed for the horizontal bearing of the anchoring walls at heel and toe, where the soil is well confined, and also for the footings at the eastern portion of the dam, where the foundation material is of manifestly better character.**

The results of the foregoing tests of the safe bearing value of the soils at the site are given in Table 5. The following comments apply to the table:

1.—The variations among the results are characteristic of the foundation soils in question.

2.—In view of the fact that the areas under load in the several tests were very small, it is reasonable to assume that under the actual conditions of loading the foundation soil would show less yielding for a given load. Generally speaking, the more confined the soil under compression, the less its yielding under load, and hence the higher its safe bearing value; furthermore, the greater the area under compression, the greater the proportion of the stressed soil which is confined against "flow".

3.—In general, the foundation soil of the east bank at the dam site has a higher bearing value than that of the middle of the valley, or of the west bank.

*Loading of the Foundation Soil.*—Calculations as to the loading of the foundation soil were based on the assumption that where the cut-off, whether in its original state or underpinned, is of relatively great depth, and especially where it extends down to bed-rock, the cut-off wall forms a relatively rigid, unyielding support for the dam; whereas the foundation soil down stream from the cut-off forms a yielding support. For practical purposes, therefore, it has been assumed that, when under load, the dam pivots about the top of the original concrete cut-off wall, the cut-off and the foundation soil down stream therefrom each carrying its due proportion of the total vertical load. The axis about which the dam is assumed to pivot is approximately in the plane of the base of the original footings. Following this assumption further, it is evident that the deflection in the foundation soil would vary approximately uniformly from zero at about the center of the cut-off wall to a maximum at the down-stream edge of the toe of the dam.

There is, of course, a certain amount of flexibility in the concrete footings; that is, the true deflection (and hence stress) diagram of the soil underlying the structure would not be a straight line. However, the effect of such flexibility in the footings is indeterminate, and the straight-line assumption was adopted in computations.

TABLE 5.—RESULTS OF TESTS OF BEARING VALUE OF FOUNDATION SOILS AT STONY RIVER DAM SITE.

| Experiment No. | Character of soil under test. | Yielding of Soil, in Inches. Load, in Pounds per Square Foot. | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 000 | 2 000 | 3 000 | 4 000 | 5 000 | 6 000 | 7 000 | 8 000 | 9 000 | 10 000 | 11 000 | 12 000 |
| 1..... | Black loam................................ | ...... | ...... | 0.004 | 0.018 | 0.040 | 0.079 | 0.093 | 0.100 | 0.138 | 0.152 | 0.156 | ...... |
| 2..... | Sandy yellow clay......................... | 0.002 | 0.018 | 0.045 | 0.086 | 0.093 | 0.141 | 0.168 | 0.177 | 0.179 | 0.186 | 0.206 | 0.230 |
| 3..... | Hard yellow clay containing small stones | ...... | 0.007 | 0.020 | 0.029 | 0.045 | 0.066 | 0.074 | 0.080 | 0.117 | 0.153 | 0.248 | ...... |
| 4..... | Brown clay................................ | ...... | ...... | 0.003 | 0.009 | 0.060 | 0.133 | 0.218 | 0.300 | 0.372 | 0.498 | ...... | ...... |
| 5..... | Yellow clay............................... | ...... | ...... | 0.007 | 0.008 | 0.021 | 0.028 | 0.043 | 0.056 | 0.070 | 0.078 | 0.103 | ...... |
| 6..... | Black gumbo............................... | ...... | 0.027 | 0.058 | 0.102 | 0.159 | 0.231 | 0.313 | 0.407 | 0.500 | ...... | ...... | ...... |
| 7..... | do. | ...... | 0.001 | 0.015 | 0.041 | 0.074 | 0.091 | 0.094 | 0.119 | 0.155 | 0.206 | 0.244 | ...... |
| 8..... | Yellow clay, containing pebbles........... | 0.006 | 0.031 | 0.046 | 0.090 | 0.117 | 0.170 | 0.184 | ...... | ...... | ...... | ...... | ...... |
| 9..... | do. | ...... | ...... | ...... | ...... | ...... | ...... | 0.153 | 0.158 | 0.162 | 0.167 | 0.173 | 0.181 |

NOTE.—The figures showing the yielding were taken from plotted results of the tests. The area of soil under load was approximately 3.6 sq. in. in all experiments, except that in No. 6 the area under load was 35.2 sq. in. In no case was the soil under load confined against lateral "flow."

The results stated below were derived on the basis of the further assumption that the original, as well as the strengthened, footings were "continuous;" in other words, that the footing for each buttress was 15 ft. wide. As a matter of fact, this assumption is hardly warranted, in so far as the original structure is concerned, as will be discussed in more detail later herein. Considering the loads to be carried by the buttress footings alone, the maximum soil pressures for the original structure would be approximately double those stated.

Referring to the typical sections of Plate V, the computed pressures on the foundation soil, in the case of the original structure at Bay 35, are as follows:

I.—Under the "most severe conditions within the limits of reason", as set forth in Column (5), Table 3, with the exception that uplift pressure is considered to be negligible, the maximum vertical pressure on the foundation soil is 3 560 lb. per sq. ft.; the corresponding slant pressure is 6 000 lb. per sq. ft.

The foregoing assumptions, as also those in the three cases following, treat the horizontal thrust as taken up entirely by the frictional resistance of the foundation soil at the base of the footings. If the uplift pressure of Column (5), Table 3, were to be taken into account, the result would be to diminish the previously stated maximum soil pressures somewhat, but not to a considerable extent.

II.—If, for the sake of comparison, one similarly applies to Bay 35 of the original structure the more severe conditions which, according to Column (9), Table 3, are assumed to apply to the same section of the strengthened structure, the maximum vertical pressure on the foundation soil becomes 3 930 lb. per sq. ft.; and the corresponding slant pressure becomes 6 740 lb. per sq. ft.

The maximum soil pressures at the maximum height sections of the original structure are approximately the same as those indicated above for the typical section.

Due largely to the use of the new toe-wall (Plate V), the unit compressive loads on the foundation soil were reduced considerably by the reconstruction, despite the fact that under the "most severe

conditions" the strengthened structure is considered to be subject to higher loading than the original structure. Thus:

III.—Under the "most severe conditions within reason", as set forth in Column (9), Table 3 (but disregarding any uplift pressure), the maximum vertical soil pressure for the strengthened structure at Bay 35 is 3 560 lb. per sq. ft.; the corresponding slant pressure is 4 795 lb. per sq. ft.

The latter pressure should be compared with that of 6 740 lb. per sq. ft. derived for the original structure under the same assumptions of loading (Case II). If, in Case III, uplift pressure had been assumed to be present, the results would not have been altered materially, because, in view of the assumed pivoting about the cut-off wall, uplift pressure under the "heel" would tend to increase the maximum compressive load on the foundation soil, whereas uplift pressure under the original footings down stream from the cut-off would tend to decrease such maximum load. Had the new "heel" been assumed to take up a portion of the horizontal thrust, the result would have been to increase the maximum vertical, but to decrease the maximum slant, soil pressures.

IV.—Under the "normal maximum load" conditions of Column (7), Table 3, the maximum vertical soil pressure for the strengthened structure at Bay 35 is only 3 105 lb. per sq. ft.; and the corresponding slant pressure is only 3 550 lb. per sq. ft.

It is probable, therefore, that, in view of the strengthening of the original structure, the foundation soil under the higher portions of the structure will not be subjected to loads in excess of 4 000 lb. per sq. ft.

In the design of the footings of the new spillway section, between Buttresses 10 and 19, the fact that the footings rest directly on shale bed-rock made the problem comparatively simple, and detailed discussion appears to be unwarranted. In that section the maximum unit compressive load is approximately 5 tons per sq. ft., whereas, in the writer's opinion, the foundation shale can take at least 20 tons per sq. ft. in compression.

*Strength of Footings.*—Apparently, it was intended by the designers of the original structure that the load should be transmitted

into the foundation soil through the buttress footings only. Throughout the larger part of the structure the buttress footings are 7 ft. 6 in. wide. The intervening space, also 7 ft. 6 in. wide, between buttress footings, was apparently intended to be simply floored over with reinforced concrete, varying from 8 to 12 in. in thickness. Such flooring, however, was connected rigidly by reinforcing steel to the buttress footings, the heavier lower fiber reinforcement being cut at the center of each bay. In the study of the strength of these footings, in connection with the safe bearing value of the foundation soil, alternatives submitted themselves, neither of which afforded satisfactory results.

In the first place, if the buttress footings were considered to carry the entire vertical load, then manifestly the unit loads would be twice as great as those derived on the assumption that the footings are continuous across the entire 15-ft. width of the bays. For instance, referring again to the typical sections of Plate V, such doubled maximum vertical soil pressure would become approximately 7 120 lb. per sq. ft., and the maximum slant pressure would become approximately 12 000 lb. per sq. ft., at the down-stream edge of the footings of the original structure under the "most severe conditions within the limits of reason". In the case of the strengthened typical section, such doubled maximum vertical and slant soil pressures would become approximately 6 090 and 8 190 lb. per sq. ft., respectively, under the "most severe conditions". The latter pressures, however, unlike those previously stated, are found at the up-stream edge of the original toe; for it is hardly conceivable that the new toe-wall, which is continuous for the sections of maximum height between old and new spillways, could act otherwise than as a continuous footing, in so far as the transmission of vertical load into the foundation soil is concerned.

The original footings, if assumed to extend only 3 ft. beyond each buttress, were approximately strong enough to carry the doubled unit loads just mentioned, but the adopted safe bearing value of the soil, *viz.*, approximately 5 000 lb. per sq. ft., would evidently be exceeded. Hence, it was imperative that the vertical load on the foundation soil should be distributed over a width of 15 ft., and not merely 7 ft. 6 in., for each buttress.

Turning then to the second or alternative assumption, it is readily seen that only by the use of continuous footings was it possible to keep the loads on the foundation soil within safe limits. As a matter of fact, even in the original structure, some load must necessarily have been carried by the intermediate flooring, inasmuch as it was strictly a part of the adjacent buttress footings. The main buttress footings could not take load (and hence deflect, by reason of compression of the underlying soil) without transmitting part of such load by shear into the adjacent flooring. It is probable, however, that in the downstream portions of the original footings such action caused stresses considerably in excess of the safe working stresses for concrete and steel.

Moreover, in the original portion of the structure, continuity of footings was necessary in order that the footings might act in harmony with the continuous horizontal, or upper, member of the new anchoring wall at the toe. The original footings, however, were not strong enough for this purpose. The weakness lay, not merely in the intermediate reinforced concrete flooring, but also in the main buttress footings themselves for, under these conditions, the length of the cantilever (measured from the face of the buttress toward the center of the bay) becomes 6 ft. 9 in., instead of 3 ft., as under the former alternative assumption. Thus, in the case of the typical section (Plate V) the allowable vertical soil pressure on the footings, with assumed working stresses of 700 lb. per sq. in. for the concrete and 18 000 lb. per sq. in. for the reinforcing steel, would be only about 1 700 lb. per sq. ft.

The adopted method of strengthening the footings is shown in the typical sections of Plate V. It was accomplished by adding both concrete and reinforcing steel. The object was to enable the original intermediate flooring to act as a part of the main footings, which would thereby become continuous, and also to strengthen the original buttress footings. In view of the relatively short clear span of such continuous footings (about 13 ft. 6 in.), it appears reasonably certain that in transmitting the load into the foundation soil the footings act in part as inverted arches. It seemed proper to utilize such arch action in the design, and therefore the arbitrary, but not unreasonable, assumption was made that one-third of the load would be carried by the footings acting as arches and the remainder by the footings

acting as continuous slabs or beams. Such continuous beams have tension in the lower fibers at the buttresses, but have compression in the lower fibers at the centers of the bays.

Concrete was added on top of the original footings adjoining the bases of the buttresses, so as to make more effective the existing steel in the bottom of the footings; whereas, near the middle of the bays, the new concrete serves as a matrix for the new transverse reinforcing steel (bent as shown in section *A-A* of Plate V). The new concrete was well tied and bound into the old concrete, both by steel dowels extending through the base of each buttress and into the original buttress footings, and also by the roughening of all surfaces of contact. This roughening was done partly with picks, by hand, but mainly by the use of air-hammers. After roughening, the surfaces were thoroughly cleaned and made wet before applying the cement grout coating which immediately preceded the placing of the new concrete.

The new footing concrete, however, has two other functions: First, it serves to embed the tie-steel leading down stream from the anchoring wall at the heel, and hence it was extended to the rear of the bays, even though there the original footings did not require reinforcement. Secondly, it serves to transmit the horizontal thrust from the buttresses and buttress footings into the tie-steel.

The approximately triangular section of concrete, shown in Section *B-B* of Plate V, at the junction between deck and footings, has been referred to in connection with the description of the new anchoring wall at the heel, serving as it does to take up the reaction of the upper cantilever member of the "heel". The concrete of this triangular section was usually poured in connection with the new "heel", and prior to the placing of the new footing concrete.

At Bays 27 to 33, inclusive, extra depths of new concrete were added over the original footings. This was done partly in order to add strength by adding weight to these sections of maximum height. The quantities of concrete involved, however, were so great that, for the sake of economy, it was deemed advisable to leave a number of large pockets in the concrete. These pockets were made practically water-tight, and act as tanks. From the time that these pockets were first filled with water, they have remained full. Thus the only sacrifice in weight is that due to the difference between the specific gravity of water and concrete.

Another reason for adding such extra concrete was to raise the top of the strengthened footings at least 5 ft. above the bottom of the new outlet channel (shown by Figs. 16 and 17). The outlets from the sluice-gates in Bays 30 and 31 were covered with a substantial reinforced concrete slab which completely separates the gate openings and

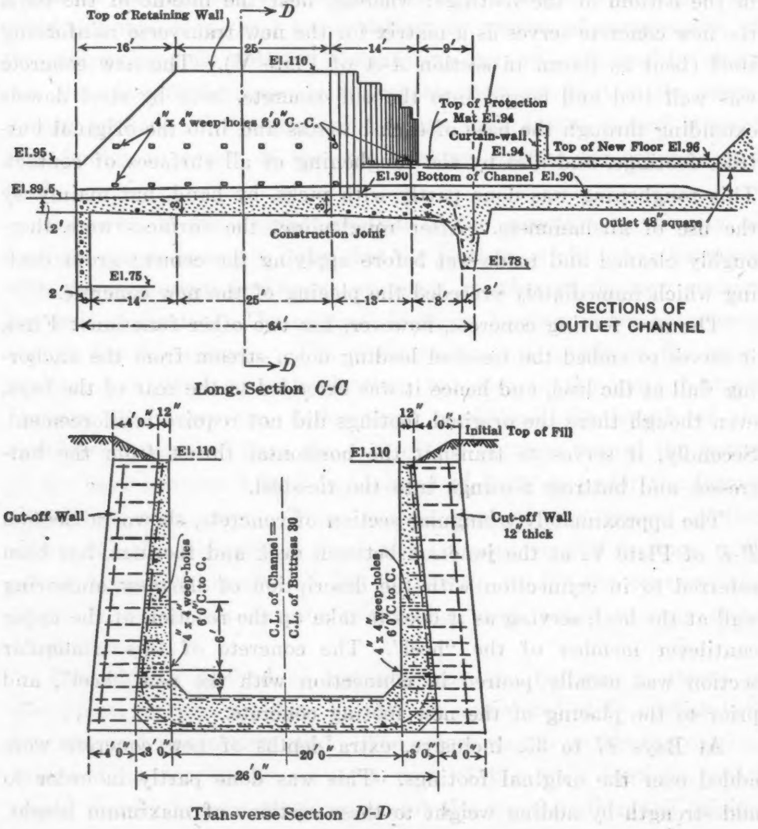



FIG. 24.

outlets from the operating mechanism. Consequently, no spray due to the opening of the gates can interfere with their operation, by freezing in winter, for instance. Moreover, it was thus possible to house in the operating mechanism of the gates entirely by carrying the slab (acting as a roof over the outlets) forward to meet the curtain-wall, as shown in the longitudinal section of Fig. 24.

At the east end of the dam, beginning at Buttress 49, it was considered allowable, as already explained, to increase considerably the unit loads on the foundation soil. Hence, in that section, the original scheme was followed of utilizing the main buttress footings to carry the entire vertical load. For this purpose the original footing required only minor strengthening. Here the possible cracking of the intermediate flooring was considered to be of minor consequence; however, the tie-steel from the new "heel", extending as it does a comparatively short distance down stream from the "heel", was well covered with concrete, doweled both into the floor and the buttress footings.

*Pressure Grouting Under Footings.*—The contact between the original footings and the foundation soil was in some places found to be faulty. Generally speaking, the original footings had not been carried into the foundation soil as deeply as might have been desired, in consequence of which certain soft spots required compacting. Also, hollow spaces were found under the footings around the 4 by 12-in. weep-holes which had been left at intervals in the footings as constructed originally. Such voids were probably due either to leakage from the reservoir or to flowing ground-water.

Moreover, it was discovered that, at least in some bays, the foundation soil had been washed away at the junction between the footings and the cut-off wall. This is shown clearly in Fig. 23 where the chalk line on the down-stream face of the original cut-off wall shows the extent of the erosion which had taken place. The photograph from which Fig. 23 has been reproduced was taken in Bay 19, but it was evident, from the conditions exposed during the wrecking of adjacent portions of the original structure, that the void in question had no connection with the break in the dam; instead, it probably communicated with a near-by weep-hole through the footings. It is believed, therefore, that this condition did not arise at the time of the failure, but had been brought about gradually by leakage through the unbonded construction joint at the top of the original cut-off wall. The construction joint would naturally open somewhat, due to the deflection of the foundation soil when loaded.

These faulty conditions, it was concluded, could best be remedied by pressure grouting. For this purpose, a Ransome-Caniff grout mixer, of about 4½ cu. ft. capacity, was used; air under pressure was obtained from a small compressing outfit and utilized at pressures varying from

25 to 100 lb. per sq. in. at the grout mixer. The mixer, in turn, was connected to pipes sealed into the weep-holes with concrete or cement grout. Mixtures of cement and sand, in the proportions of 1: 1 or 2 : 1, were used as far as possible, but, for the greater part of the work, neat cement grout was found to be more practical; the presence of sand in the grout was frequently the cause of clogging the pipe connections and consequent delay.

It was necessary to plug temporarily all weep-holes in the neighborhood of that to which grout was being applied under pressure. Observations showed that grout under pressure traveled under the footings for a distance of at least 100 ft. Inasmuch as the water contained in the cement grout helped to fill the voids, it was found necessary to adopt the process of forcing grout into the voids connecting with a given weep-hole until they refused to accept more, and then suspending grouting for a day or more. In the meantime the grout would set, leaving the uncombined water free to be blown from under the footings, on subsequent resumption of grouting, by simply unplugging temporarily a few communicating weep-holes. Such holes were then re-plugged and more grout was forced into the voids. This process was continued for each weep-hole until finally it was found that the hole would accept no more grout. After grouting had been completed and the grout had set, the weep-holes were again opened, so as to communicate with the underlying soil, by simply punching or drilling holes through the grout.

By these means, it is believed, the foundation soil under the original footings was thoroughly compacted so as to warrant the assumption that the footings are everywhere in contact with the foundation soil, either directly or through intermediate grout.

### Leakage and Drainage.

*Leakage Through Foundation Soil.*—Reference has been made to the leakage which took place through the construction joint at the top of the original cut-off wall, and also to the fact that this trouble was remedied as an incidental result of the construction of the new anchoring wall at the heel. Further leakage, however, was discovered under and through the original cut-off wall, especially where the structure is of maximum height. During the work of investigation, prior to beginning actual reconstruction, test-pits were sunk up stream from,

and adjacent to, the original cut-off wall at Buttresses *B* and 37 (Plate II). The pit at Buttress *B* was approximately 45 ft. deep, and by means of it the cut-off was investigated at its deepest section. In neither of these test pits was the cut-off wall found to be porous or the seal into bed-rock defective.

Except for a test pit sunk with great difficulty through the footings and foundation soil adjacent to the down-stream side of the cut-off wall at Bay 28, the investigation of the cut-off at the sections of maximum height was postponed until the actual work of reconstruction was being carried on. The cut-off in this region, *viz.*, between Buttresses 26 and 32, was relatively shallow, at least, as compared with the hydrostatic head due to the reservoir; and therefore it was the more important that this portion of the cut-off should be impervious. The postponement of the investigation in this region was due only to the great expense of putting down test pits so near the river bed without adequate coffer-damming.

In connection with the excavation for the new "heel", therefore, test pits were sunk at intervals, and extending from the elevation of the bottom of the "heel" down to the bottom of the original cut-off. The findings in the test pit sunk through the footings in Bay 28 had already indicated, much to the writer's surprise, that the cut-off in that region was not all that it should be, and the test pits sunk from the trench excavation for the "heel" on the up-stream side of the cut-off confirmed these findings. It was definitely established that there was easy communication for water between the up-stream and down-stream sides of the cut-off wall at Bay 28. Probably some of the water which flowed back and forth, depending on the hydrostatic pressure conditions, passed through the cut-off wall itself; but, at this point, undoubtedly the greater part of the communication was underneath, through openings between the cut-off wall and the bed-rock.

At Bay 30, also, it was evident that the seal of the cut-off into bed-rock was imperfect. In fact, at one point a layer of sandy mud, averaging ½ in. thick, was plainly visible between the concrete and the shale, indicating that, in the original construction, the trench had not been properly cleaned preparatory to the placing of concrete. Moreover, the cut-off wall itself was porous to the extent of allowing a constant trickle of water to pass through from the down-stream side. That there was any such leakage at all was the more noteworthy because

of the fact that the hydrostatic pressure on the down-stream side was due solely to the ground-water in the clayey foundation soil—though it should be remembered, of course, that the foundation soil under this maximum height portion of the dam is of poorer quality than that at any other part of the structure.

In view of the foregoing findings, it was determined to underpin the original cut-off wall for approximately 200 ft., *viz.,* from about Buttress 32 westward to the new spillway. A notch, about 24 in. deep and 18 in. wide, was cut into the bed-rock, with its center approximately at the up-stream edge of the original cut-off wall. The notch was carefully cleaned and kept free from water while it was being filled with a rich concrete which was carried to a height of at least 12 in. above the bottom of the old cut-off.

This work proved to be the most expensive part of the entire reconstruction, partly because of the constant fight with the water which flowed into the trenches through the pervious soil under the river-bed, and partly because of the fact that the character of the soil required practically continuous sheeting and bracing. So difficult, in fact, was it to carry on the work and to hold the up-stream side of the excavation for the "heel" that at Bays 30 and 31 it was found advisable first to pour the "heel" concrete and then, after that concrete had set, to tunnel under it in order to investigate the efficacy of the cut-off seal into the rock. Such tunneling was done from the adjacent bays, and, to insure solid work under the "heel" at Bays 30 and 31, grout pipes were inserted while the concrete for the cut-off underpinning was being placed, the ends of the pipes being exposed under these portions of the "heel". Later, the voids at the ends of these pipes were filled with grout, by using the Ransome-Caniff grout mixer, under a pressure of about 75 lb. per sq. in.

At certain places, as shown on Plate II, the cut-off underpinning concrete was extended on the up-stream side of the cut-off up to the elevation of the bottom of the "heel". In such cases a slip joint was made between the "heel" and the underpinning concrete. This joint consisted essentially of a longitudinal groove, about 12 in. wide and 6 in. deep, lined with tar-paper. The groove was centered on the bottom of the "heel". When the "heel" concrete was poured, therefore, a tongue of concrete, likewise 12 in. wide and 6 in. deep, was formed in the groove integral with the "heel" concrete. On all horizontal

surfaces, and on the up-stream vertical face of the groove, single sheets of two-ply tar-paper were used to separate the two bodies of concrete. On the down-stream vertical face of the groove, however, three sheets of two-ply tar-paper were used.

The purpose of the slip joint is to allow the "heel" to move down stream a fraction of an inch under the horizontal thrust acting against the dam and hence transmitted by the tie-steel into the "heel". Without such "play" an unfavorable loading might be caused at the lower end of the long cantilever member of the "heel", because of the resistance of the relatively inelastic underpinning concrete. The latter, of course, is practically immovable because of its bond into the bed-rock. The utility of the slip joint is probably to be justified on theoretical rather than practical grounds, but, in view of the small expense involved, there seemed to be no warrant for omitting this precaution.

At about the middle of Bay 26 it was found that there was a pervious spot in the bed-rock strata just below the practicable limit of underpinning. Therefore a churn-drill hole was sunk into the strata in question, and grout pipe connections were made, which, after the completion of the underpinning and the new "heel", were utilized in an attempt to force grout into the stratum. However, a pressure of 75 lb. per sq. in. did not suffice to force down an appreciable quantity of grout.

It is not improbable that the failure to force more grout into this stratum was due to the fact that there was no vent for such rock-water as might have been displaced by grout had a means of escape for such water been provided. It happened that, for the grouting of the underpinning of Bays 30 and 31, as previously mentioned, two grout pipe connections had been left, and thereafter on the work, profiting by the experience at Bay 26, at least two grout pipes were placed in each void to be filled. Usually, the second pipe was connected to the pressure grouting apparatus only after the first pipe refused to accept any more grout. In such cases it was commonly impossible to force grout into the second pipe, and apparently the main function of that pipe was to serve as a vent. In the writer's opinion, it is not unlikely that some of the reported failures of pressure grouting operations at other dam sites have been due to the fact that no such vent existed, or had been provided, for the escape

of air or ground-water contained in the voids sought to be filled by grouting.

Referring again to the faults in the original cut-off construction, which required remedying by the means just described, the reconstruction afforded a rare opportunity for observing the results of this construction work, which was probably done with average care. These observations show that great care is necessary even in foundation work, where it is too often assumed that any kind of work will suffice.

*Extension of Cut-Off*—Inasmuch as the insufficient depth of cut-off wall in the neighborhood of the break in the dam was undoubtedly the direct cause of the failure, it was advisable to pay especially careful attention to any further deficiency in depth of cut-off. At the sections of maximum height, precautions were taken in the way of underpinning the original cut-off wall, as discussed previously. Such underpinning extended westward to the new spillway which in turn covered completely the area affected by the failure. (Plate IV.) Here the combination cut-off and anchoring wall at the heel extends into the shale rock to an average depth of about 10 ft., in the manner illustrated on Plate III. The depth is greatest in the neighborhood of Buttresses 14 and 15, because there, as mentioned previously, the rock was found to be somewhat shattered.

In the same region the rock yielded a considerable quantity of water, especially after the drain holes had been drilled. Consequently, after the combination cut-off and anchoring wall had been completed, a hole was drilled, with the Calyx machine, through a pipe which had been embedded in the center of the wall at about the middle of Bay 13. The hole penetrated the underlying shale to about Elevation 70. It was intended to grout this hole for the purpose of cutting off any communication which might exist in this neighborhood through passages in the rock under the cut-off. It was assumed that the existence of any such connection would be shown, when pressure grouting was applied, through the manifestation of bubbling or other symptoms at a drain hole extending to about the same elevation and approximately 7 ft. down stream in Bay 13. However, a surprise was afforded in that the hole would accept no grout, and it was apparent that the rock-water passages, which had been tapped a short distance down stream in Bay 13, either came from a greater depth, or more probably

communicated to the westward—the slope of the bed-rock (Plate II) being upward and westward.

West of the new spillway, beginning at Buttress 10, the depth of the original cut-off was as inadequate as at the point of failure. At some places, especially near steps in the bottom, the depth of the cut-off was less than 5 ft. below the bottom of the footings (Plate II). It is probable that everywhere in this region, except possibly at one or two of such steps, the original cut-off extended to a depth of at least 6 ft. below the original ground surface. That the wall was not deep enough to cut off a pervious stratum of dark color in the overburden will be seen by referring to Fig. 7, in the upper right corner of which the bottom of the original cut-off wall is shown supported by a prop.

The entire west end of the dam, *viz.*, between Buttresses 1 and 10, was provided with a new "heel", the required depth of which varied from 1 to 5 ft. below the bottom of the original cut-off. The new "heel" was made not less than 2 ft. 6 in. wide at the bottom, this being a width convenient for trench excavation. A cut-off wall of this minimum width was then extended downward from the bottom of the "heel" to seal into the bed-rock. In this region the bed-rock consists of shale of a very good quality; in fact, in some places the rock is so hard that a notch only 6 in. deep was considered to afford a sufficiently good seal. At other places, however, the notching was carried to a depth of more than 3 ft. on account of penetrating a stratum of soft shale which in part overlies the very hard stratum.

At the bottom of the required depth of "heel" a standard slip joint was provided, in the manner previously described; and just under the old cut-off (but above the slip joint) a shoulder of concrete, with a minimum width of 12 in., was formed. The purpose of such a shoulder of concrete was to make certain that any vertical load transmitted into the old cut-off wall would in turn be taken up by the new "heel" and cut-off, thus guarding against any possible separation of the old concrete from the new and against the placing of any excessive shearing stress on the tie-steel connecting the new anchoring wall to the original structure.

As will be seen from Plate II, a core-wall had been provided in the original construction extending 30 ft. west of Buttress 1. As a part of the reconstruction, this core-wall was extended upward to

the elevation of the new parapet (Elevation 142.5) and downward to seal into the shale. Also, because of the increase in assumed high water elevation, it was extended about 15 ft. farther westward. The precautions taken in providing a cut-off against percolation at the west end of the dam were of a more conservative nature for the reason that it was near this end of the dam that the failure had occurred.

Between Buttresses 32 and 46 the depth of the original cut-off, with a maximum of 45 ft., was concluded to be sufficient in proportion to the maximum hydrostatic pressure. This conclusion was based on the fairly well substantiated assumption that in this region the original cut-off is impervious throughout its entire depth, but it did not depend on the character of the seal of the cut-off into the bed-rock. As a matter of fact, however, a personal inspection of the bond between concrete and rock as exposed by the test pits at Buttresses *B* and 37 convinced the writer that the seal is as effective as could be desired. East of Buttress 46, on the other hand, certain difficulties were encountered.

It had been assumed by the builders of the original structure that the cut-off extended to bed-rock at least as far east as the approximately vertical step in the cut-off at the middle of Bay 54 (Plate II). In fact, all records of the original work show the cut-off as being in contact with bed-rock. It is not surprising, therefore, that the results of core-drill holes Nos. 3 and 8, sunk during the investigation prior to reconstruction, were discredited when they indicated that bed-rock was reached only at a considerable distance below the bottom of the original cut-off. Nevertheless, some mental reservations were made, and subsequently a test pit was sunk at the middle of Bay 48. The findings were by no means comforting, for, after penetrating flat-top boulders, the upper surfaces of which offered some reason for their previously erroneous identification as bed-rock, the pit extended through clay deposits into strata of disintegrated coal. Rock was not reached until at about Elevation 92. It is a shale, and quite satisfactory.

In view of the discovery of the disintegrated coal, which was decidedly pervious, the existing cut-off was manifestly inadequate. This was especially true of the portion east of the middle of Bay 48. It was concluded, therefore, to underpin the original cut-off down to a line sloping with approximate uniformity from Elevation 92 at Buttress 46 to Elevation 108 at Buttress 53, and meeting the bottom of the original

cut-off at the latter point. The proviso was made, however, that the disintegrated coal should be cut-off wherever its top came within 1 ft. below the bottom of such sloping line. The probable genesis of the disintegrated coal and the measures taken to cut it off are shown on Plate II. Drill holes Nos. 9 and 10 were sunk after the discovery of the disturbing conditions just described.

It was also concluded to underpin the original cut-off east of the middle of Bay 54, as well as the core-wall which originally extended 50 ft. east of Buttress 56. Excavation for the underpinning in these two localities proved to be very difficult because of the existence of a sandstone talus in the form of huge boulders. The most economical method was found to be the sinking of shafts from the surface of the ground and the construction of tunnel drifts leading from such shafts under the original cut-off. Some of the crevices discovered between the boulders were found to be filled with clay, and others were open. In one instance a crevice at least 2 in. wide was traced for 6 ft., and it extended even beyond that distance. The shattered rock was very difficult to excavate and required frequent blasting.

Before the excavation was filled with concrete, grout holes were churn-drilled into the rock, and grout pipes were placed in crevices so that later, when these were pressure-grouted, it is probable that an impervious rock, concrete and grout curtain was formed, extending between Buttresses 54 and 56 down to at least Elevation 110. Similarly, grout pipes were left in the higher portions of the tunnel drifts so as to insure, later, the filling, by pressure grouting, of any air pockets left in the higher sections. The same grout pipes served to allow the escape of confined air while the main concrete was being placed. The concrete was carried to a considerable height in the shafts previously mentioned and, because of its slushy consistency, undoubtedly created considerable pressure throughout the drifts. Pressure-grouting was not applied until the concrete had set for a day. The east core-wall was extended approximately 24 ft. farther east. Both the old and new portions of the core-wall were built up to Elevation 142.5, to correspond with the top of the new parapet.

*Drainage.*—In the original structure the intermediate flooring of each bay had 4 by 12-in. weep-holes. A number of these holes existed at each edge of the main buttress footings, the shorter dimension of the holes being parallel to the footings. The weep-holes were in

pairs, one hole at each edge of the intermediate flooring; the pairs were about 8 ft. from center to center, though the spacing was closer at the up-stream ends of the bays. As a matter of fact, a considerable number of these weep-holes were found to be clogged with concrete which had evidently flowed under the weep-hole forms at the time the original footing or flooring concrete was being placed. Apparently, it had not been thought necessary to provide deeper drainage for the foundation soil.

In planning the reconstruction of the dam, however, it was concluded to provide more positive means of drainage. The system adopted for that portion of the original structure which remained intact differed from that adopted for the new spillway section:

*1.—Drainage of Foundation Soil.*—In the preliminary plans for reconstruction, the writer proceeded on the theory that the more drains the better, and especially the more positive, the relief of uplift pressure. Consequently, it was proposed in general to extend three drains per bay into the foundation soil. These drains were to consist of perforated pipes such as used in the finally adopted system of drainage. The up-stream drains were to be inclined, and thus were to extend to within about 5 ft. of the down-stream face of the cut-off wall and to a depth of about 5 ft. below the bottom of the new "heel". The middle and down-stream drains of each bay were to be vertical, at distances of about 20 and 30 ft., respectively, from the cut-off wall. At the seven bays of maximum height it was proposed to place the up-stream drains 7½ ft. from center to center transversely, thus providing two up-stream drains per bay. It was also proposed to keep all the original weep-holes open through the original concrete and the concrete of the footing strengthening.

On the other hand, the finally adopted system of drainage, though utilizing the same methods as proposed in the preliminary plans, differed as to the number and location of drains. It was based on two main considerations, namely:

(*a*) The necessity of relieving serious uplift pressure; but, also,

(*b*) The inadvisability of facilitating percolation through the foundation soil.

It was concluded that deep drains might be harmful in earth foundations under circumstances where they would not be harmful

in rock foundations, because, in the former, they might actually facilitate the flow of water by providing artificial channels where previously there was no channel at all. Especially might such results follow if the deep drains were to extend close to the bottom of the cut-off wall.

A consideration of secondary importance was the effect of the low values of the coefficient of frictional resistance assumed in the reconstruction design. Manifestly, a given uplift pressure reduces the margin of safety against sliding by a less amount where there is a low coefficient of frictional resistance than it does where the coefficient is relatively high. It was believed to be preferable, therefore, to take such risk as might result from the existence of pockets, as it were, of unrelieved uplift pressure near the down-stream side of the cut-off wall rather than to invite further leakage by tapping such relatively harmless, deep-lying pockets of water under pressure.

In consequence of these considerations, a system of drainage was adopted which involved but a single deep drain per bay for the higher sections of the intact portions of the original structure, resting as they do on clayey foundation soil. The positions of the drains are indicated in the typical sections of Plates II and V. From the former it is seen that the deep drains for the foundation soil extend from Bays 20 to 47, inclusive. The drains consist of 3-in. wrought-iron pipes, perforated with four ⅜-in. holes per linear foot, one hole in each quadrant. Pipe drains, rather than unlined holes, were adopted for the foundation soil, in order to make the drains relatively permanent, and also because unlined openings would not resist erosion of the foundation soil by leakage water.

The deep drains are about 20 ft. down stream from the cut-off wall, except at the bays of lesser height, where they are at a minimum distance of 12 ft. from it. The drains ordinarily extend to a depth of 5 ft. below the bottom of the new "heel", though they were limited to a depth equalling two-thirds of that of the cut-off, measured from the bottom of the original footing to the bottom of the underpinning, if any, of the original cut-off. They were sunk in lengths of 12 ft. or less. Flush-joint piping was used, so as to avoid couplings or other projections; the latter might leave an annular space around the drains to serve as an outside channel for flowing water without the protection of a metal lining.

Great difficulty was experienced in sinking the drain pipes, primarily because of the boulders encountered in the foundation soil. Various methods were tried, among them the process of jetting, that is, of sinking a drain pipe, open at the lower end, by using a jet of water under pressure playing on the soil inside the pipe near its lower end, so as to allow the pipe to be forced down by hammer blows at the top.

The most satisfactory method, however, and that generally adopted, involved the use of a pile-hammer. For this purpose a McKiernan-Terry, No. 2, pile-hammer was placed on the job and operated either by air or by steam at a pressure of about 80 lb. per sq. in. In order to facilitate driving, the blacksmith on the work fitted the drain pipes with solid points made from pieces of old steel shafting, approximately 3 in. in diameter. The points were welded inside the ends of the pipes, with results as shown in Fig. 19. This figure shows also the perforations in the pipes. It is to be admitted that the perforations were probably clogged with clay to a certain extent as the pipes were sunk. On the other hand, any considerable hydrostatic pressure near the drain pipes would soon open the perforations.

A simple wooden cap prevented injury to the pipe, and the flush-joint pipe did not prove so weak at the joints as to buckle or otherwise cause trouble. In a number of instances, however, boulders proved to be impassable obstacles, and in such cases pipes were driven at near-by locations. In each instance the drain pipes were sunk through the weep-holes of the original structure. The process was necessarily slow. Sometimes it required 12 hours, and even 24 hours, to drive a pipe to a depth of 20 ft.

In general, the deep drain pipes were encased in grout or concrete through the footings and for a depth of about 18 in. below the footings, with the object of keeping the water percolating through the deeper foundation soil separate from leakage water manifesting itself immediately under the footings, and finding its origin perhaps through the cut-off wall. Thus the deep drains are enabled the better to perform their function as detectives, so to speak. The upper ends of the drain pipes were threaded and left exposed for at least 3 in., so as to allow the pipes to be capped or to be connected to pressure gauges. The pipes, of course, were thoroughly cleaned out after they were driven. Very few of the deep drains showed leakage after the work of reconstruction was completed, and none showed any considerable leakage.

Further, under the adopted system of drainage, the two up-stream pairs of weep-holes in each bay were plugged with concrete in connection with the pouring of the triangular section of concrete at the junction between deck and footings. About one-half of the remaining weep-holes—distributed as uniformly as practicable—were left open.

As the reservoir refilled during the latter part of the work of reconstruction, leakage appeared at weep-holes in Bays 51 and 36. At first it was thought that possibly the leakage at Bay 51 was caused by ground-water, perhaps from the water being fed into drill hole 10 (Plate II) which was being sunk at the time. However, by coloring the reservoir water immediately up stream from Bay 51 with potassium permanganate, it was finally proved that the leakage came through the cut-off. Probably it was due to inadequate bond along a horizontal construction joint in the new "heel", within a foot or two in elevation of the old construction joint at the top of the cut-off wall. The leakage was practically stopped by the simple expedient of dumping clay into the reservoir over the new "heel" from a float extending out from the east shore.

It is probable that in time the leak at Bay 51 would have sealed itself, as has since become true of the leak at Bay 36. In the latter instance the same remedy was applied as at Bay 51, and fine cinders, as well as clay, were dumped from boats into the reservoir water over the "heel". The cure, however, was not effected in this manner; but it was noted that the leakage was gradually decreasing, even though the water level of the reservoir grew higher, until by November 1st, 1915, there was hardly any leakage at this point.

If the leakage at these two points was due to carelessly treated construction joints in the new "heel", the stopping of the leakage may in turn have been due—in part, at least—to the taking of some of the horizontal thrust by the new "heel", with consequent tightening of the construction joint by reason of the compression caused in the up-stream portion of the "heel".

Other leaks, of a minor nature, which appeared in the original portions of the structure have stopped or are stopping themselves in the same manner. Thus on May 25th, 1915, there were eighteen leaks or traces of leakage from weep-holes or drain pipes in the foundation soil, whereas, by November 1st, 1915, there were only three, and these the attendant designated as "traces". In no case

has a pressure test indicated a head of more than 2 ft. above the tops of the weep-holes or drain pipes.

At times certain of the drainage openings yielded muddy water, instead of the normally clear water. In the opinion of the writer, these phenomena, usually temporary, were due to adjustments in the foundation soil caused by increases in loading as the reservoir refilled. In a few instances such openings were grouted shut, rather than take any chances of harmful erosion under the footings.

It may not be amiss to refer here to another phenomenon, namely, that in a number of pipes which are sealed into weep-holes and communicate through the footings into the foundation soil, the water level remains practically at the top of the pipes, which are about 2 in. in diameter and extend about 6 in. above the top of the concrete footings. These pipes do not actually weep, except in the sense that when water is dipped out of the pipes, they refill in a day or two, but again do not overflow. The possibility is indicated that capillary tension over the surface of the ground-water rising in the pipes is sufficient to prevent overflow, but the writer has been unable to advance any explanation which is satisfactory to himself.

*2.—Drainage of Rock Foundation at New Spillway.*—In the case of the shale forming the foundation for the buttress footings of the new spillway, there is, of course, no danger that deep drains will facilitate erosion of the foundation. This is especially true because of the fact that the shale contains practically no lime which might be taken out in chemical solution. Moreover, as described in detail under the heading "Geological and Foundation Conditions", the rock contains water-bearing seams, and it was possible that ground-water communication might exist between the reservoir and certain springs which appeared in the area excavated for the new spillway footings. Hence it was concluded to provide more drains at the new spillway than in the foundation soil under the intact portions of the original structure.

The system of drainage provided at the new spillway consists in general of three deep drains per bay, placed as shown in the typical sections of Plate III. The drains were sunk with the Calyx, shot-drilling apparatus. In the bed-rock they remain unlined. Above the shale, however, 4-in. pipes were sealed as well as possible into the rock and carried up at least to Elevation 96, in order that leakage

might be detected above the level of the water which ordinarily stands in the new spillway bays as high as the elevation of the horizontal drain through the foot of the spillway apron (Plate III). It was practically impossible to prevent the continuously flooded condition of the space between the new spillway footings; at the same time, this condition has the merit of preventing disintegration of any shale by exposure to the atmosphere.

The up-stream drain of each trio extends down to Elevation 70, well below the bottom of the combination cut-off and anchoring wall. The middle and down-stream drains extend down only to Elevation 75, which elevation also is below that of the bottom of the cut-off. The number of drains was increased at Bays 11 and 12, being the two most westwardly bays of the new spillway; and, inasmuch as the strata slope upward toward the west bank, it is probable that the rock-water, following the dip of the strata, will be intercepted at these bays.

Ever since the deep drains of the new spillway were sunk there has been a flow of rock-water in Bays 12, 13, 16, and 19. However, both the quality of the water and its relatively constant yield prove that there is little, if any, direct communication between such rock-water and the reservoir. The reservoir pressure doubtless has some influence on the rock-water in question, for there was a minor increase in the yield as the reservoir gradually refilled. Yet it is also true that during "dry" periods the rock-water yield has decreased somewhat, even though during the same periods there was but little fluctuation in reservoir level. The maximum rock-water pressure, measured at the tops of the new spillway drains, was equivalent to a head of about 2 ft. The drains overflow freely into the surrounding pools of water.

In view of the drainage system provided throughout the higher portions of the structure, it appears reasonable to assume that, if the drains are not frozen shut at times when uplift pressure may be accumulating under the footings, no serious uplift pressure is to be feared. The assumptions of uplift pressure under the "most severe conditions within the limits of reason", as set forth in Table 3, are based on the drainage provision above described.

*Curtain-Wall and Roof.*—It was primarily to prevent any considerable freezing at the top of the weep-holes and deep drains throughout the higher bulkhead sections of the dam that a curtain-wall and roof were constructed, as illustrated in the typical section *B-B* of Plate V.

Manifestly, an ice plug, say, 6 in. deep, at the top of a weep-hole or drain pipe, would for practical purposes serve to confine water under the footings as effectively as though the openings had been filled with concrete and made a part of the footings.

It is believed that, as the result of housing in all bays having deep drains, namely, Bays 11 to 47, inclusive (Figs. 16 and 17), the temperature of the corresponding footings will be prevented from falling to more than a few degrees below the freezing point. During cold weather the radiation of heat outward through the curtain-wall and roof will be largely counterbalanced by the radiation upward of heat from the unfrozen foundation soil and by the further radiation of heat from the reservoir water through the deck of the dam into the enclosed space. Water is at its greatest density at about 39° Fahr., with the density gradually decreasing as the temperature decreases to the freezing point. Consequently, water in the reservoir under an ice covering is at least several degrees above freezing temperature; the difference may be as great as 7 degrees. The assumption made as to the effect of the curtain-wall and roof is supported by observations said to have been made at various hollow dams which are provided with both deck and apron, thus being entirely enclosed. It is reported that at some such structures freezing does not occur in the enclosed portions.

As soon as practicable in the late winter of 1914 the writer had maximum and minimum thermometers placed in the old spillway section of the Stony River Dam, and had the larger openings into that section closed so as to allow a series of records to be made, covering exterior and interior temperatures. Conditions were not appropriate for conclusive results, because the reservoir, of course, had been emptied at the time of failure, thus exposing both apron and deck to the cold, with the result that the only source of heat was the foundation soil under the old spillway. Nevertheless, even these observations showed clearly that variations of temperature inside the enclosed spillway section were far less than the corresponding variations in outside temperature; also, that periods of relatively higher and lower temperature inside the structure lagged considerably behind the corresponding periods of relatively higher and lower temperature outside.

During the winter of 1914-15 the lowest temperature within the enclosed portions of the dam was + 26° Fahr., and the lowest exterior

temperature was — 12° Fahr. Under these conditions ice formation has taken place, but not of a serious nature. At the very least, the enclosing of the bays enables the attendant to inspect carefully the drainage system, regardless of weather conditions, and also protects the operating mechanism of the outlet gates. When ice forms within the enclosed portions, the attendant can readily keep it broken or cut away so as to prevent any harmful results.

A portion of the curtain was constructed in the form of a roof, instead of continuing the wall vertically to intersect the deck, in order to avoid enclosing what would have been a triangular section at the top, exposed to cold air on the down-stream side and frequently to the ice covering of the reservoir on the up-stream side. For the bulkhead section between the old and new spillways, the curtain-wall was made 12 in. and the roof 7 in. thick, and, for the eleven bays immediately east of the old spillway, the thickness of the curtain-wall was reduced to 6 in., the roof remaining 7 in. in thickness. At the base of each curtain-wall there is an opening, about 12 by 18 in., fitted with a wooden flap door, as shown on Plate V, so as to allow the outward flow of any leakage water, but at the same time to keep air currents and snow out of the enclosed space. Wooden ladders bolted to the buttresses are provided between the walkway and the floor at each enclosed bay.

In addition to this primary purpose, the curtain-wall and roof furnish effective lateral stiffening for the buttresses. The writer does not suggest that the bracing of the buttresses in the original structure was insufficient, yet he confesses that he feels better satisfied with the buttresses braced by the curtain-wall, especially as regards the lower portions of those of maximum height, where originally an additional lower set of horizontal brace-beams might have been desirable.

Furthermore, the curtain-wall tends to equalize deflection in the foundation soil under the footings, and, consequently, to shift the load from weaker to firmer soil. Finally, the curtain-wall was utilized in a minor way in the design of the strengthening of the intermediate flooring, allowing, as it did, the omission of transverse reinforcement for a reasonable distance on each side of it.

Excepting for these secondary functions of the curtain, it would have been well to make the wall and roof hollow, for instance, by the

use of cement plaster on metal lath. A hollow construction could have been obtained with concrete, of course, but at a sacrifice of economy. Hollow construction would have better conserved the heat available for warming the enclosed portions of the bays. Relative to this it is interesting to note that more freezing has taken place in the eleven bays immediately east of the old spillway, where the curtain-wall is only 6 in. thick, than in the remainder of the enclosed bays, where the wall is at least double that thickness.

*Leakage in Superstructure.*—At the time of failure there were pronounced leaks at certain contraction joints in the deck. In the type of construction used the deck slabs are not in contact with each other at the buttresses. Instead, a tongue, integral with the buttresses and varying from 18 to 24 in. in width, separates the slabs. Contraction joints exist, therefore, between these tongues and the slabs, over all buttress haunches. It appears that, in the original construction, insufficient care was taken in forming these joints, with the result that in most cases they did not fulfill their intended functions, and contraction cracks occurred only at intervals averaging about 75 ft. The tenacity of the bond in certain of the intended joints is well illustrated in Fig. 5 which shows the deck of Bay 18 overhanging westwardly for a distance of about 13 ft., the adjacent Buttresses 17 and 16 with their intermediate deck having fallen away due to the undermining of the buttress footings. Naturally, such joints as did act opened excessively and allowed considerable leakage to take place.

The worst leakage of this kind occurred on the east side of Buttress 29, where, during the severely cold weather obtaining at the time of the failure, ice formed to such an extent as to block the stairway leading from the walkway down to the outlet gate-operating mechanism. It was considered impracticable to attempt to reconstruct all the contraction joints so as to make them perform properly their intended function. Instead, all joints which were acting too freely were made as nearly water-tight as possible by cleaning them out thoroughly, packing them with tar-paper placed on edge, and then pouring in hot asphalt. The asphalt has a tendency to flow from the joints during extremely hot weather, but it is believed that such asphalt as remains in the joints, together with sediment deposited by leakage water, will prevent any serious leakage. As a matter of fact, the joint at Buttress 29 still leaks somewhat, especially in cold weather. However, by

reason of the bay being enclosed by the curtain-wall and roof, the attendant can now with little difficulty clear the stairway of the trifling quantity of ice that forms during severely cold weather.

In the new spillway section precautions were taken to insure positive action of the contraction joints by covering the entire surface of such joints with two thicknesses of three-ply tar-paper, or the equivalent thereof. The leakage here has been inconsequential.

## Miscellaneous Problems.

*Strength of Decks and Buttresses.*—In the case of the greatest flood reasonably to be anticipated, the deck of the original structure would be subjected to greater loading than that for which it was designed. The resulting increase in stresses would be greatest at the bottom of the upper lift of the deck, namely, at Elevation 126. Here the maximum stresses with head-water at Elevation 142.25 would be approximately 22 000 lb. per sq. in. in the reinforcing steel and 800 lb. per sq. in. in the concrete. The original steel is reported to have been purchased under a guaranty of 55 000 lb. per sq. in., minimum elastic limit, and the concrete was of a character warranting the assumption of an ultimate strength of at least 2 000 lb. per sq. in. in 90 days.

Under these conditions, it did not appear necessary or advisable to strengthen the decks of the original structure. Likewise, it was not considered necessary to strengthen the buttresses, other than to give them the benefit of the incidental stiffening provided by the curtain-wall previously described.

At the new spillway section, on the other hand, it was considered proper to design the deck for a loading due to head-water at Elevation 142.25, notwithstanding the fact that the new spillway deck thus has greater strength than the deck of the original portions of the structure. Also, the thickness of the buttresses was increased, as compared with the original structure. Above Elevation 112 the increase in thickness was a minimum of 4 in., and below that elevation a minimum of 6 in. Furthermore, the new buttresses are more heavily braced laterally than in the case of the original construction. The walkway slab is thickened so as to serve as a brace-beam, with its principal dimension horizontal, and two other brace-beams, with their principal dimensions vertical, are provided lower in the structure. Detailed comparisons may be made by examining the data on Plates III and V.

*Increase in Storage Capacity.*—In view of the much larger spillway capacity provided in the reconstruction, it appeared logical to make temporary use of some of the depth over the spillway by providing increased storage capacity. It was concluded to be feasible to utilize in this way approximately 3 ft. of the 6 ft. 6 in. available depth over the main spillways. Such use for storage purposes, however, is temporary only in the sense that, during extraordinary floods, the entire depth over the main spillways must be available for flood discharging purposes. The resulting increase in the available storage capacity was approximately 380 000 000 gal., or about 25% (Fig. 10), and it afforded a certain consolation to the owner for the heavy expenditures entailed by the failure and reconstruction of the dam. It may reasonably be said that, even in the case of the original structure, it would have been possible similarly to utilize some of the 3 ft. difference in elevation between the crest of the spillway and that of the bulkhead sections; yet it is equally true that an appreciable quantity of additional storage could have been obtained only at the risk of overtopping the structure during floods.

*Flash-Board Supports.*—The problem involved in increasing the available storage capacity by about 25% was to provide a temporary dam, in other words, flash-boards, which would fail within comparatively narrow, predetermined limits of loading. Thus, it was imperative that 3-ft. flash-boards should not be overtopped by more than 3 ft. 6 in. at the very maximum. The effect of the use of such 3-ft. flash-boards on the flood discharging capacity of the spillways was carefully considered, and a number of hypothetical cases were worked out. One of these cases, previously discussed in this paper, is shown in Fig. 14, wherein it was assumed that, with the greatest flood reasonably to be provided for, the 3-ft. flash-boards might not fail until the head-water had reached an elevation 5 ft. above the base of the flash-boards. The result of such studies was the conclusion that the use of flash-boards would not affect seriously the maximum flood discharging capacity of the dam as reconstructed.

It was impracticable to provide at small cost reliable means which would cause the flash-boards to fail with head-water 3 ft. over the boards, and yet not fail at lower elevations. On the other hand, it is highly undesirable that the flash-boards should fail under any except the more severe freshets; otherwise the inconvenience of renewing

the flash-boards and of the loss of the 3-ft. depth of stored water would occur too frequently.

Both of these considerations were subservient to a third condition, namely, that the means of lowering the flash-boards or causing them to fail must be absolutely automatic, inasmuch as conditions warranted the employment of only a single attendant, and at the critical time he might be absent or asleep, or might for any other reason not attend to the flash-boards when necessary. Finally, the cost of the flash-board provision and of replacing the flash-boards was necessarily to be made as low as possible.

For the purpose of meeting the foregoing requirements, the writer had about seventy experiments made on steel bars or "pins" to serve as flash-board supports. The experiments were intended chiefly to find a material which would fail at a predetermined stress, but would not allow the pins to bend considerably at lower stresses. It was also desired that, when failure occurred, it should consist in the pins snapping off at approximately the elevation of the spillway crest, thus allowing the crest to be cleared absolutely of all obstruction by the flash-boards and their supports. The experiments were performed in the outlet channel, where, by using the sluice-gates and the previously accumulated head in the partly refilled reservoir, it was possible to duplicate the heads which would exist under actual operating conditions, and hence use flash-boards and supports of the same size as under actual conditions.

A controlling condition was the fact that flash-board sockets had been provided in the original spillway 3 ft. 6 in. from center to center. The same spacing was used for the crest of the new spillway, so as to allow interchange of equipment. The scope of the tests covered various conditions of head, lengths of time during which heads were maintained, and degrees of turbulence (akin to reservoir wave action), as well as various kinds and conditions of steel. Inasmuch as it was desired to avoid the use of pins which would bend, and because of previous experience with wrought-iron pins, it was not thought worth while to spend time in testing pins of that material. However, steel pins were tested, including soft and hard steels. The highest carbon content was approximately 1.40 per cent.

None of the steel pins tested in their natural condition gave satisfactory results. Those made of steel with even as high as 0.90 to

1.10% carbon (railway spring steel) persisted in bending before breaking, whereas steels with higher carbon content, though they did not bend unduly, were erratic as to the head or load at which they broke. This was especially true of tool steels. Practically, as a last resort, the writer tried the process of hardening the steel; that is, the pins were heated and then hardened by plunging into cold water. Even under these conditions satisfactory results were obtained only within comparatively narrow limits. The softer steels, of course, would not harden, and the very high carbon steels became the more erratic in their behavior by reason of such hardening. Round pins were used, because of the uniform resistance of pins of circular cross-section, no matter at what point the load might be applied (thus making it unnecessary, in this regard at least, to exercise care in placing the pins).

The most satisfactory results were obtained by the use of open-hearth steel with a carbon content varying between 0.40 and 0.80% and phosphorus and sulphur contents each not exceeding 0.05 per cent. In the hardening process the pins were heated to a "cherry red" before quenching in cold water. The heated portion extended about 8 in. above and below that point in the pin which when in place would be at the spillway crest. As the result of the tests on hardened specimens with such characteristics, it was concluded that 1¼-in. round pins, with carbon content of 0.62 to 0.77%, aiming at 0.72%, and sulphur and phosphorus contents limited as above, would fail at approximately the predetermined point, *viz.*, when the elevation of the head-water was about 4.3 ft. above the main spillway crests, or about 1.3 ft. above the top of 3-ft. flash-boards. It was also concluded that such pins would not be subject to excessive temporary or permanent deflections before breaking.

As it was impracticable to wait for further tests before making the final selection of the supporting pins to be used, two sets were ordered for each spillway, together with a sufficient number of extra pins to allow for making confirmatory tests. Subsequent analysis showed that the pins delivered actually have a carbon content of 0.68 per cent. The flash-board pin sockets in both old and new spillways were adjusted by grout filling to a uniform depth of 18 in. The pins were cut to 4 ft. 6 in. lengths, so as to allow them to project 3 ft. vertically above

the spillway crests. Seasoned spruce planks were used as flash-boards, with an actual net height of 2.78 ft. above the spillway crests.

The final tests, made on pins chosen at random from those purchased for actual service, showed exceedingly satisfactory results. In no cases where the conditions were equivalent to those of actual service did the pins fail at heads of less than 4.0 ft., nor did they longer than momentarily withstand without failure heads of more than 4.6 ft. above the base of the flash-boards. Moreover, the maximum temporary deflection above the top of the sockets, with the head-water at elevations slightly less than that at which failure occurred, was only about 10° down stream from the vertical. In those instances where the head-water was allowed to subside to zero after having very nearly, but not actually, attained the predetermined elevation, the pins showed a permanent deflection or set of less than 3° from the vertical. The results of these tests, made in the outlet channel, showed surprisingly little difference between the effects of turbulent and of still head-water.

The apparent brittleness of the hardened pins is the more remarkable in view of the "qualifying" test to which the pins had been submitted after they had been hardened, *viz.*, they had been dropped from the crest of the dam to the concrete flooring of the outlet channel, a distance of more than 50 ft. In order to make doubly certain that defective bars were eliminated, each pin was dropped twice in this manner. Sometimes they fell across previously dropped pins, thus concentrating the impact at a single point of the pin. Yet very few which were put through this conclusive test were thereby broken. This "qualifying" test was applied, not merely to the pins used for the confirmatory tests in the outlet channel, but to all pins which were placed in actual service.

With regard to the sensitiveness of the pins in question, the results of an earlier test may be of interest. In that test $1\frac{3}{16}$-in. round pins, with carbon content of 0.62%, were used at the regular intervals of 3 ft. 6 in. to support flash-boards about 38 in. high. The head-water was maintained for $10\frac{1}{2}$ hours at an elevation approximately 3.7 ft. above the base of the flash-boards. At the end of that period the head was increased to 4.2 ft., and within a few minutes two of the three pins under test, supporting three panels of flash-boards, snapped off simultaneously at the top of the sockets.

Since the reconstruction of the dam has been completed, the head-water, with the flash-boards in place, has reached Elevation 139.83, which is about $12\frac{5}{8}$ in. over the top, and 3.83 ft. above the base, of the flash-boards. The supporting pins met the expectations, as none failed, despite the fact that they were under severe load for several hours. Subsequently, the flash-boards were taken down for the winter, and it was found that the pins had suffered no permanent deflection.

*Underpinning Problems at the New Spillway.*—The original footings of Buttress 10, which was left intact and forms the west abutment of the new spillway, were about 22 ft. higher than the footings of the adjacent new Buttress 11, and about 27 ft. higher than the footings of new Buttress 12. The footings of the latter buttresses rest on bed-rock, whereas the original footings of Buttress 10 rested on the clayey over-burden. Evidently, it was necessary to provide secure support for Buttress 10 as well as to insure the stability of the foundation soil under the adjacent (up-hill) portion of the original structure. This was accomplished by underpinning Buttress 10 with a retaining wall founded on bed-rock. In this manner it was possible, in effect, to extend the footings of Buttress 10 down to rock and, at the same time, prevent the lateral displacement of the soil under the adjacent footings. The retaining wall was extended down stream as a portion of the new spillway channel, and decreases in height as the channel swings away from the west bank. It is of reinforced concrete, and is of substantial proportions for a sufficient distance down stream from the footings of Buttress 9 to confine adequately the foundation soil immediately down stream from that portion of the dam, as shown by Figs. 16 and 17.

The type of construction of the underpinning at Buttress 10 is illustrated in Figs. 25 and 26, which show the underpinning in successive stages of construction. To avoid the necessity of temporary shoring under Buttress 10, relatively narrow excavations were first made for the 4-ft. wide counterforts, leaving the intermediate soil to take the weight of the buttress and its overhanging footing (which was not removed). Due to the character of the soil in question, it was impracticable to allow the buttress to be supported in this manner for any considerable length of time. Fortunately, no severe rains occurred in the meantime, and the counterforts were constructed without mishap up to the bottom of the footings. Then, by means of holes

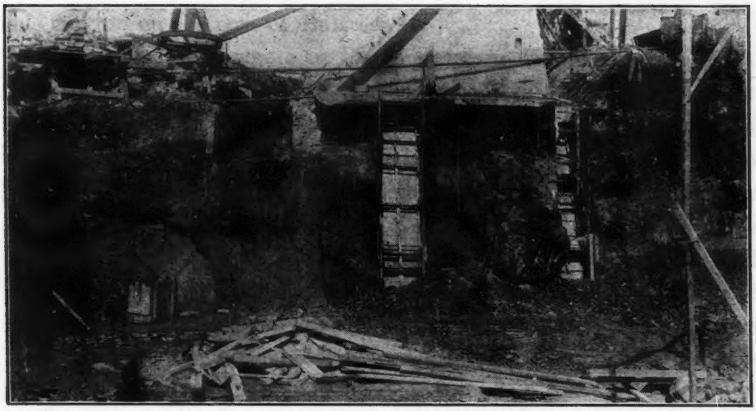


FIG. 25.—UNDERPINNING AT BUTTRESS 10.

FIG. 26.—UNDERPINNING AT BUTTRESS 10, STONY RIVER DAM.

drilled through the footings, grout was forced between the surfaces of contact. Subsequently, the counterforts took the load without noticeable sinking on the part of Buttress 10.

As soon as the concrete in the counterforts had set sufficiently, a reinforced concrete panel, 4 ft. thick, was constructed, bearing against shoulders which had been left in the counterforts. The concrete in this panel was poured directly against the clayey bank, so as to prevent the formation of voids and consequent settlement. All voids left immediately under the footings were filled with grout, not so much to enable the west half of the Buttress 10 footings to transmit load into the foundation soil as to prevent lateral displacement of the soil underlying Buttress 9 and its footings. The retaining wall panels were provided with weep-holes in order to drain adequately the supported soil.

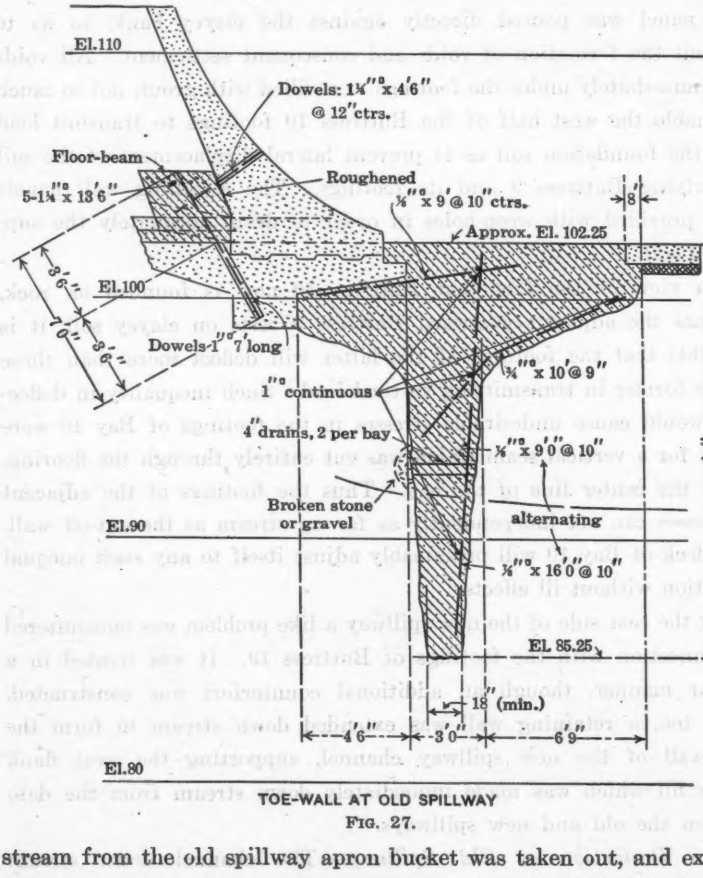
In view of the fact that Buttress 10 now is founded on rock, whereas the adjacent westward Buttress 9 rests on clayey soil, it is probable that the footings of the latter will deflect more than those of the former in transmitting vertical load. Such inequality in deflection would cause undesirable stresses in the footings of Bay 10 were it not for a vertical seam which was cut entirely through the flooring, along the center line of the bay. Thus the footings of the adjacent buttresses can act independently as far up stream as the cut-off wall. The deck of Bay 10 will presumably adjust itself to any such unequal deflection without ill effects.

At the east side of the new spillway a like problem was encountered in connection with the footings of Buttress 19. It was treated in a similar manner, though an additional counterfort was constructed. Here, too, a retaining wall was extended down stream to form the east wall of the new spillway channel, supporting the west flank of the fill which was made immediately down stream from the dam between the old and new spillways.

*Toe Protection at Old Spillway.*—The channel down stream from the old spillway has a concrete mat 9 in. thick, the mat having been placed on crushed stone which covered the original ground surface to a depth of 4 in. In the reconstruction it was felt that, considering the possible effects of frost, in the way of heaving the channel mat, and of scour at the down-stream end of the mat, such

construction could hardly be considered permanent, and it appeared to be wise to make more substantial provision.

This took the form of additional toe protection, rather than of doing away with the original channel mat and substituting a relatively expensive, heavier mat. The principal features of such toe protection are shown in Fig. 27. The row of 10 by 10-ft. mat slabs immediately down



TOE-WALL AT OLD SPILLWAY

FIG. 27.

stream from the old spillway apron bucket was taken out, and excavation was made for a toe-wall extending to a depth of 17 ft. below the top of the channel mat or hearth. This toe-wall was designed to act as an anchoring wall, similar in this respect to the anchoring walls provided for the bulkhead portions of the original structure. However, it was carried to a greater depth, in order to prevent the undermining

of the main footings of the old spillway in case the channel mat should be washed away in whole or in part.

Here, again, the toe-wall forms, in effect, an extension of the original footings. Its load is received through the original toe of the spillway. However, because the apron of the spillway had been cast separately from the footings and the buttresses, it was impossible to utilize the apron, as originally constructed, to transmit any load into the toe-wall, except in so far as the weight of the apron and the frictional resistance, along the surfaces of contact between the apron and buttresses, were concerned. It was found advisable to utilize the apron to a greater extent by constructing a so-called floor-beam, as shown in Fig. 27, and tying the apron into the floor-beam with the steel dowels there shown. This arrangement also served the purpose of distributing the load on the base of the buttress over a great distance (up and down stream), thus reducing the unit load on the concrete in proportion.

In addition to the deep toe-wall, wing-walls were extended down stream, as shown in plan on Plate IV. The inner end of each wing-wall extends to the same depth as the toe-wall, but the outer ends are approximately 11 ft. deep below the top of the mat. The wing-wall at the west end of the old spillway was capped by the concrete of a wasteway, which allows any excessive head of water in the channel immediately down stream from the bays of maximum height to relieve itself into the old spillway channel. The new fill between spillways has been protected on its east flank along the old spillway mat by the construction of a wall of sandstone boulders laid in cement mortar.

*Anchoring Wall at Heel of Gate Sections.*—At Bays 30 and 31 the openings from the reservoir to the outlet gates prevented the construction of the standard type of "heel" shown on Plate V. The upper cantilever member of the anchoring wall would have interfered with the opening. Therefore, a modified method of construction was used, as shown on Plate VI. In this case the anchoring wall extends from the top of the concrete protection mat, immediately up stream from the gate openings, to a depth of about 15 ft. below the original footings. The cantilever arms are not so long as in the standard "heel", for the reason that the tie-steel enters the anchoring wall at the middle, inclining downward from the original footings. The steel bars find embedment in the new concrete immediately adjacent

to, and bonded into, the lower portions of Buttresses 29, 30, and 31, as shown in Section *B-B* of Plate VI. The bars were encased entirely in concrete to protect them against corrosion in the foundation soil.

In order to place the tie-steel, holes were opened by blasting through the original cut-off wall, and tunnels were carried upward under the footings to connect with openings which had been cut down through the footings. The bars were then placed and the concrete poured from the interior of Bays 30 and 31. The special construction at these bays was simpler, in so far as the anchoring wall itself was concerned, but was more expensive than the standard type because of the necessity of placing the tie-steel in the manner described.

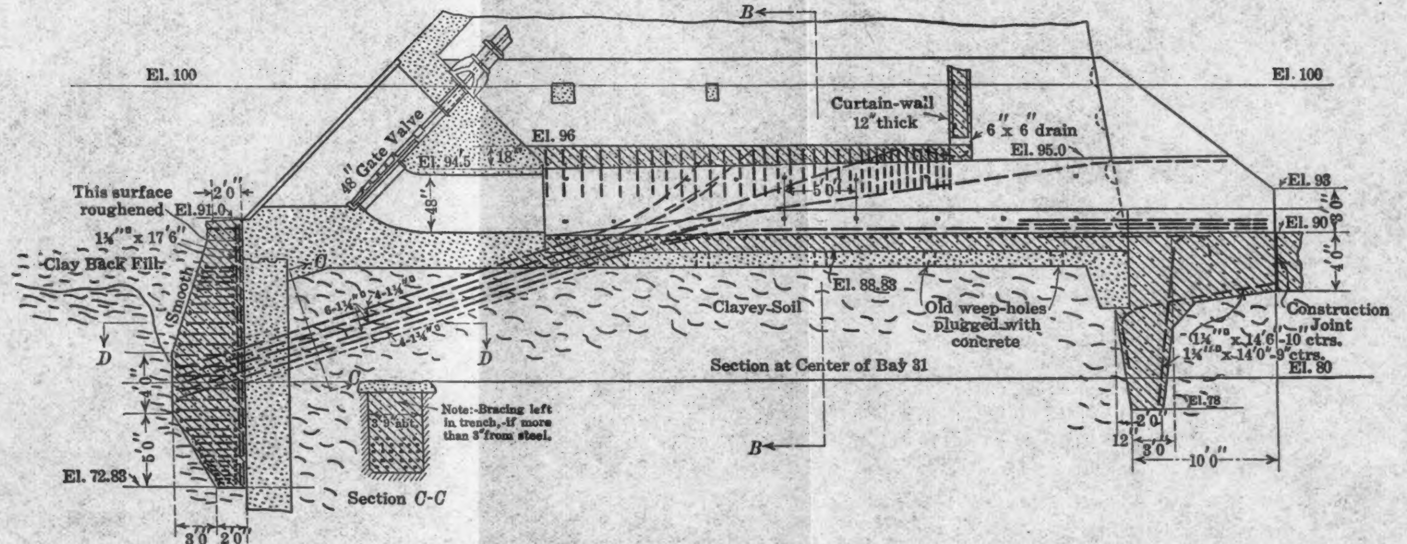
*Outlet Channel.*—The outlet channel has been referred to and partly described in preceding portions of the paper. Its purpose is essentially to prevent any scour which might result either from discharge from the sluice-gates or from water flowing over the intermediate spillway which extends the length of the bulkhead section between the old and new main spillways. The sides of the channel are constructed as retaining walls to hold the down-stream fill. Longitudinal and transverse cross-sections of the outlet channel are shown in Fig 24, and Plate IV shows it in plan. The channel extends a total distance of 64 ft. down stream from the toe of the original structure, and is protected by a cut-off, about 12 ft. deep, penetrating a bed of heavy boulders. The details of this structure are not such as to warrant further description.

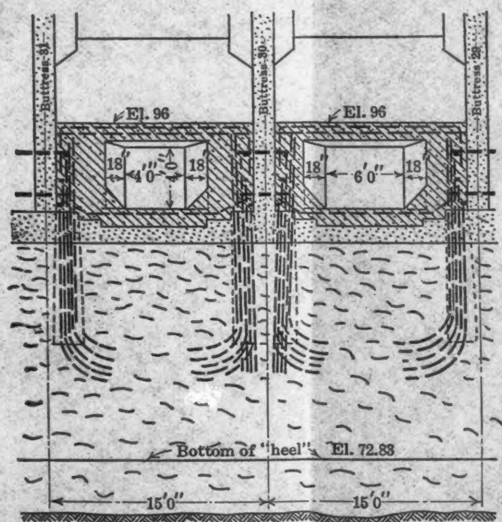
A connecting channel was necessarily constructed to lead from the outlet channel directly down stream to the old river-bed. This change of stream channel has a further advantage in that it protects the lower end of the old spillway channel mat from the scour, due to sluice-gate discharge, to which it was subjected under the original construction.

## Materials of Construction.

*Cement.*—The cement for the work of reconstruction, as also that for the original construction, was furnished by the Alpha Portland Cement Company, and all the cement for the reconstruction came from its Lehigh Valley mills. The cement was required to meet the Specifications of the American Society for Testing Materials plus those specifications of the United States Government (Bureau of Standards)

PLATE VI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1917.
SCHEIDENHELM ON
RECONSTRUCTION OF
STONY RIVER DAM.




Transverse Vertical Section *B-B*

ANCHORING WALL
AT HEEL OF
OUTLET GATE BAYS.

Note:-Shaded portions indicate new work.
All reinforcing and tie steel bars are twisted square.

pertaining to insoluble residue and loss on ignition. It was tested at the mills by representatives of the Pittsburgh Testing Laboratory.

*Stone.*—Crushed sandstone was used for the concrete aggregate. It was quarried from ledges exposed along the mountain side within convenient distance of the work. Three quarries were opened; two of these required short hauls on the temporary, standard-gauge railroad, and the third furnished rock to the crushers directly by small cars operating on a narrow-gauge track. The cost of quarrying was heavy, inasmuch as the ledges were in no case of great depth, and the cost of stripping was therefore proportionately high.

In view of the relatively high cost of the materials for concrete, sandstone boulders were utilized to as great an extent as possible. These boulders were ordinarily of one-man size, and either were found in the river-bed or were yielded by the excavation for anchoring walls, cut-off extensions, etc. The larger proportion of the boulders was found in the excavations near the middle of the valley.

*Sand.*—As in the case of the original construction, sand was obtained by crushing and rolling the native sandstone, though the product was not the best that could be desired. Probably the chief defects in the sand were its inferior grading, and especially the large proportion of fines. Nevertheless, the resulting concrete was acceptable, as shown by tests made from time to time on standard specimens cast in most cases from concrete taken directly from the mixer in the regular course of the work. The specimens were shipped to Pittsburgh to be tested.

*Steel.*—Open-hearth reinforcing steel was furnished by the Carnegie Steel Company, under the June 1st, 1912, Standard Specifications, of the American Society for Testing Materials, for cold-twisted, steel reinforcing bars. Thus the steel was required to show a minimum yield point of 55 000 lb. per sq. in. The standard design data, as used for the cold-twisted bars, may be of general interest, and are therefore submitted in Table 6.

*Classes of Concrete Construction.*—Four standard classes of concrete construction were used in the reconstruction, and, on the basis of these classes, working stresses for use in design were specified, as follows:

*Class A-1.*—Reinforced concrete; proportions $1:2\frac{1}{4}:3\frac{3}{4}$; allowable working stress for steel, 18 000 lb. per sq. in.; for concrete,

TABLE 6.—STANDARD DESIGN DATA FOR COLD-TWISTED SQUARE STEEL BARS.

| Diameter or size of bar, in inches. $d$ | * Minimum embedment, surface of concrete to center of bar, in inches. | † Lap or Length of Embedment Required to Develop Maximum Safe Unit Stress in Bar, in inches—$L$. At 16 000 lb. per sq. in. Bond stress, in pounds per square inch. 75 ($A$-2) | At 16 000 lb. per sq. in. 67 ($B$) | At 16 000 lb. per sq. in. 60 ($C$) | At 18 000 lb. per sq. in. 75 ($A$-1) | $10d$ Radius for bend, in inches. | † Lengths—see Diagram. $a$ In inches. | $b$ In inches. | ‡ Allowable range of spacing between centers of bars, in inches. | Preferable spacing between centers of bars, in inches. |
|---|---|---|---|---|---|---|---|---|---|---|
| ⅜ | 1½ | 20 | 23 | 25 | 23 | 4 | 12 | 8 | 1½ to 6 | 2 |
| ½ | 2 | 27 | 30 | 34 | 30 | 5 | 16 | 11 | 2 to 6 | 2½ |
| ⅝ | 2½ | 33 | 38 | 42 | 38 | 6 | 19 | 13 | 2½ to 10 | 3 |
| ¾ | 3 | 40 | 45 | 50 | 45 | 7½ | 24 | 16½ | 3 to 12 | 4 |
| ⅞ | 3½ | 47 | 53 | 58 | 53 | 9 | 28 | 19 | 3½ to 13 | 4½ |
| 1 | 4 | 54 | 60 | 67 | 60 | 10 | 32 | 22 | 4 to 14 | 5 |
| 1¼ | 4 | 67 | 75 | 84 | 75 | 12½ | 40 | 27½ | 4½ to 16 | 6 |

NOTE.—Use no ⅜-in. bars except under especially approved conditions.

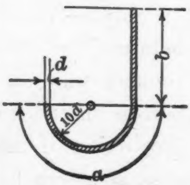
Use ½-in. bars only where unavoidable, *e. g.*, for slabs only 6 or 8 in. thick.

As far as possible use bars of same diameter in same portion of structure, so to avoid confusion.

Wherever possible, bar lengths are to be specified to nearest ½ ft.

Bars should not be specified more than 40 ft. long; and ordinarily not more than 30 ft. long.

Use no clamps for splicing unless there is no other recourse.




*These dimensions apply likewise to distance from end of bar to nearest concrete surface. Where bottom of slab is cast on ground, allow 1 or 2 in. extra depth of embedment.

†Where available length for straight embedment is less than 90% of standard length, use 180° bend. (See diagram.) Letters in parentheses refer to classes of concrete.

‡Bars, in general, should not be farther from center to center than the distance from outer compression fiber of concrete to center of steel.

700 lb. per sq. in. This was used almost solely in the footing strengthening, because the protected condition of the concrete there warranted taking advantage of the economy resulting from the highest practicable working stresses.

*Class A-2.*—Reinforced concrete; proportions 1 : 2¼ : 3¾ (the same as those for the preceding class); allowable working stress for steel, 16 000 lb. per sq. in.; for concrete, 500 lb. per sq. in. This class of construction was used for the deck, apron, and brace-beams of the new spillway; also for all reinforced concrete retaining walls.

*Class B.*—Reinforced concrete; proportions 1 : 2¾ : 4¾. This class of construction was used where the design was not limited to the bare requirements measured by allowable working stresses,

and where the quantities of concrete were ample. It was used, for instance, in the buttresses of the new spillway, the anchoring walls at heel and toe of the original structure (excepting the toe-wall at the old spillway), curtain-walls and roofs, main parapet of the outside bulkhead sections, floor of the outlet channel, and the heavy mat of the new spillway channel.

*Class C.*—Plain concrete; proportions $1:3\frac{1}{2}:5\frac{1}{2}$. In general, this class of construction had no reinforcement, except for temperature stresses and the like. It was used for the buttress footings and the combination anchoring wall and cut-off of the new spillway, the counterforted retaining walls underpinning Buttresses 10 and 19, all cut-off wall extensions and the toe-wall at the old spillway. In the latter case, especially, as well as in certain other places, considerable quantities of boulders were incorporated in the Class *C* concrete. It was specified that boulders should not constitute more than 40% of the total volume, and that they should not interfere, or be in contact, with reinforcing steel.

## Construction Methods and Equipment.

The total quantity of concrete involved in the reconstruction and strengthening work was approximately 10 160 cu. yd., according to the plans; actually, about 11 700 cu. yd. (including boulders) were placed, the discrepancy of about 15% being due principally to over-run in trench excavation. The quantity of concrete reported to have been placed during the original construction of the dam was 11 800 cu. yd. Evidently, the reconstruction was by no means a small matter as compared with the original construction. The quantities involved in each case were less than those which would have been required had a dam of the solid masonry type been constructed; yet the complexities involved in, and the attention to detail required by, the type of construction actually used were very much greater than would have been necessary in the case of a solid dam.

The construction methods were essentially like those used in the original construction work, and described elsewhere.* In view of that fact, it does not seem proper to occupy space in this paper with any detailed description of the construction methods. As a matter of fact,

* *Engineering News*, January 22d, 1914.

certain special methods of procedure applied in the reconstruction have already been described in connection with the discussion of the governing features of design.

*Construction Equipment.*—As soon as practicable after the writer had been engaged to investigate the foundation conditions and determine the feasibility of reconstructing the dam so as to be safe, work was begun on a standard-gauge railroad connection. This was made, as in the case of the original construction, with the standard-gauge logging railroad of the Parsons Pulp and Paper Company, but at a point nearer the dam site and about 16 miles from Dobbin, W. Va., on the Western Maryland Railroad. The connecting spur added a little more than 2 miles to the distance. It was constructed at a cost of about $3 000 per mile for grading the road-bed, ballasting, cross-ties in place, and laying rails, though not including the cost of the rails.

This low cost was due to the fact that, in the construction of this spur, no pretense was made of keeping curvature and grades within ordinary limits, nor was much attention paid to ballasting the road-bed. The maximum curvature was 31° 15′, and the maximum grade was 12.4 per cent. For a considerable distance the grade averaged 9.8 per cent. The contour of the ground was followed closely, and the greatest part of the work of grading consisted in taking out boulders with which the ground was strewn. Small streams were bridged with tree-trunk stringers or cribbing.

The rails were new, 85 lb. per yd., and were carried on heavy ties, which, however, were spaced at greater than standard distances. After the reconstruction was completed, the rails were taken up and transferred for permanent use by the owner in lumbering operations.

Despite the inferior character of the roadbed, no accidents worthy of note occurred, and the results were such as to justify what might have been an inadequate expenditure for railroad connection, had it not been for the relatively heavy rails. The owner supplied a 42-ton geared engine from his West Virginia lumbering operations, and three of his own standard 40-ft. flat cars which were used principally for transporting rock from the quarries to the crushers. The engine was able to handle one standard loaded box car at a time up the maximum grade. Fortunately, the grade favored in-bound freight.

A Marsh-Capron mixer, of $\frac{3}{4}$ cu. yd. capacity, steam-engine driven, mixed the concrete and delivered it into buckets of $1\frac{1}{2}$ cu. yd. capacity,

set on flat cars. These cars were run on a narrow-gauge track from the mixer out to a point near the west abutment of the dam and directly under a 3-ton Lidgerwood cableway. The cableway was of 1 000 ft. span, and was used in turn to deliver the buckets of concrete either to hoppers and chutes for distribution laterally and under the deck of the original structure, or, in the case of the new spillway buttresses, into wooden bucket cars which ran on portable, narrow-gauge tracks along the top of the buttress forms, and thus allowed the concrete to be deposited near the ends of the forms. The cableway was also of general utility in shifting equipment and forms, transporting reinforcing steel, etc.

Steam was furnished both from a main plant, consisting of two 100-h.p. boilers, on the west hillside just above the spur railroad track, and also from smaller, individual boilers placed at different parts of the work from time to time, according to immediate needs. Such boilers were especially necessary during cold weather because of the condensation in the long pipe line from the main boiler plant. The walkway through the original structure, and continuing through the new spillway, served as a convenient carrier, on which were placed water, steam, and compressed air pipes which were tapped wherever necessary. Water was supplied from a duplicate set of steam plunger pumps which were set up in Bay 30 and took water from a sump fed from the river by the 20-in. sluice-gates.

An 8 by 10-in. compressor proved to be a great utility, especially in connection with the very convenient Ingersoll-Rand "jack-hammer" drills. The latter were used to drill holes in the original concrete, for the purpose of embedding steel dowels, or for blasting—as was especially necessary in cutting out the base of the deck to allow the passage of the tie-steel from the anchoring wall at the heel. These air drills were also used for roughening the original concrete wherever necessary. Operated by steam, they proved only fairly satisfactory for drilling purposes in the stone quarries, but were displaced later by a standard Sullivan steam drill.

For pumping, an 8-in. centrifugal pump and a portable, gasoline-engine driven, 4-in. diaphragm pump, were used as much as possible, but in the deeper trenches steam siphons were used.

*Cold-Weather Concreting Precautions.*—Concrete was placed throughout the winter of 1914-15 under temperatures as low as 10°

Fahr. The principal precautions to prevent concrete from freezing were: First, the water used in mixing the concrete was heated so that the temperature of the mixed batch, after being away from the mixer several minutes, was about 55° to 75° Fahr. The water was heated by live steam to a temperature considerably higher than that last mentioned because the aggregate absorbed much heat from the mortar of the batch, despite the fact that steam pipes were used in the sand and crushed stone supply bins to heat these materials also.

Secondly, the concrete or earth surfaces against which new concrete was to be poured were thawed out or warmed. This was accomplished with a jet of live steam which at the same time served to clean thoroughly the surfaces of old concrete.

Thirdly, the freshly poured concrete was covered with canvas. The canvas covering was usually arranged so as to leave an air space which, under especially severe temperatures, or with thin concrete, was warmed with coal-burning salamanders or gasoline torches. At times, also, live steam was turned in under the covering until the concrete had attained a sufficient set.

As the result of such precautions, little concrete was frozen. Those surfaces which were frost-bitten were usually in places where the concrete was relatively massive, and precautions had consequently been relaxed.

*Cost.*—Despite the fact that approximately the same quantity of concrete was placed in the reconstruction and strengthening as in the original construction, the later work was done at a considerably less total cost, due principally to the fact that much of the new concrete was comparatively massive. Nevertheless, the new work required more attention and care than if the work had been entirely of a standard nature and free from the conditions imposed by the existence of the original structure. Only about 25% of the total cost of the reconstruction and strengthening was expended in rebuilding (as a new spillway) the portion of the dam between Buttresses 10 and 19. Had this portion been rebuilt as a bulkhead section, rather than as a new spillway, the proportion of the total cost involved therein would have been even less. The remainder of the cost was expended in strengthening the original structure and in providing features which had not at first been provided. A considerable quantity of work, especially in the form of

cut-off wall underpinning, was not anticipated at the beginning of the reconstruction.

Due to the facts that the new work was scattered over the entire site and that it was carried on through the winter, the unit costs were necessarily high. The fact that the work did not conform to standards (except in the case of the new spillway superstructure, which constituted a small proportion of the work) also contributed to such high unit costs. For the same reason, the unit costs are not of general value. Thus the monthly average costs for excavation, exclusive of charges for plant and overhead expenses, varied from $0.51 for new spillway channel excavation to $7.68 per cu. yd. for excavation required for underpinning the original cut-off near the center of the valley. The corresponding costs for concrete varied from $3.40 per cu. yd. in place for Class *C* concrete in cut-off extensions to $13.43 per cu. yd. for Class *B* concrete in curtain-wall and roofs.

The results attained in the reconstruction and strengthening of the Stony River Dam could have been secured at far less cost—undoubtedly less than half the cost of reconstruction—had the features involved been incorporated in the original construction.

## Personnel.

In his capacity as Consulting Engineer in charge, the writer made inspection trips to the work about twice each month, after the reconstruction had been begun in earnest. Mr. D. N. Showalter represented the writer as Resident Engineer throughout the entire reconstruction. Mr. C. W. Hotaling was Superintendent of Construction, having filled the same position for the Ambursen Hydraulic Construction Company after it took over the original construction work in March, 1913.

The writer desires to take this opportunity to express his appreciation of the whole-hearted co-operation of Messrs. Showalter and Hotaling, as well as of the executive officers of the West Virginia Pulp and Paper Company, owner; also to acknowledge the valued advice of Daniel W. Mead and C. V. Seastone, Members, Am. Soc. C. E., with whom he was privileged to consult on several occasions during the period of reconstruction.

[illegible] and undergrowth [illegible] transpiration.

Due to the fact that the [illegible] and also [illegible] varied [illegible], the [illegible] necessarily higher. The fact that [illegible] in the case of the [illegible] small proportion of the [illegible]. For the same reason, the [illegible] [illegible] readings [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible].

The results [illegible] [illegible] [illegible] [illegible] [illegible] in the [illegible] [illegible].

[illegible]

In the [illegible] [illegible] the work [illegible] [illegible] Dr. [illegible] [illegible] [illegible] [illegible] W. [illegible] [illegible] [illegible] [illegible].

The writer desires to take this opportunity to express [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] period of [illegible].

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# SUGGESTED CHANGES AND EXTENSION OF THE UNITED STATES WEATHER BUREAU SERVICE IN CALIFORNIA

REPORT OF A COMMITTEE OF THE SOUTHERN CALIFORNIA ASSOCIATION OF MEMBERS.

## Discussion.*

BY MESSRS. GEORGE S. BINCKLEY AND CHARLES H. LEE.†

GEORGE S. BINCKLEY,‡ M. AM. SOC. C. E. and CHARLES H. LEE,‡ ASSOC. M. AM. SOC. C. E. (by letter).¶—The Committee is gratified to find that the discussion of the paper has been constructive, and has covered all sides of the subject. The Committee has been actuated by a sincere desire to get at the truth and to be of constructive service. The discussions are indicative of similar motives. Messrs. Binckley and Lee.

Although none of the Weather Bureau officials has contributed formally to the discussion of the paper, which is a matter of regret, yet the members of the Committee have had informal personal discussions relative thereto with the present Chief of the Weather Bureau, Professor C. F. Marvin, and the official in charge at Los Angeles, Dr. Ford A. Carpenter. These gentlemen, though not always in accord with the conclusions reached, assisted in many ways in amplifying the Committee's information. The members of the Committee desire to express their appreciation of the helpful suggestions and the co-operative attitude of these gentlemen toward the work.

The Committee understands that, at the request of the Cleveland Association of Members, the Local Forecaster, Mr. William H. Alex-

* Discussion of the paper by George S. Binckley, M. Am. Soc. C. E., and Charles H. Lee, Assoc. M. Am. Soc. C. E., continued from November, 1916, *Proceedings*.

† Authors' closure.

‡ Los Angeles, Cal.

¶ Received by the Secretary, January 16th, 1917.

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ander, of Cleveland, Ohio, addressed that organization on the subject of its paper. The Committee has not been furnished with a copy of this address, but was kindly loaned one by Dr. Ford A. Carpenter, from his office files.

Before proceeding further, the Committee wishes to make clear three points in which the paper may have misled those who read it.

First, the title and presentation of data might indicate that the Committee's findings and suggestions applied particularly, or even solely, to the State of California. This was far from the Committee's intention, however, as it is recognized that California enjoys its share —and a very full share—of the effort and funds at the disposal of the Weather Bureau, and that the local Weather Bureau officials are doing all in their power to obtain and distribute the information desired by the engineers of the State. The conditions set forth in the paper were regarded as typical of all the Western States, and also the Central, Eastern, and Southern States. California was chosen for detailed investigation because of the greater degree of first-hand knowledge thereof possessed by the Committee. That there is equal need for greater study of rainfall in the region east of the Rocky Mountains is indicated by the strong editorial which recently appeared in one of the leading engineering journals.*

Second, the Committee does not wish its advocacy of improvement in the mountain climatological work of the Weather Bureau to be interpreted as supporting the idea that precipitation data collected at properly located mountain stations will yield all the information as to water supply and its variations needed for the design of hydraulic structures and projects. On the contrary, the Committee is in full accord with Mr. N. C. Grover's statement, that stream-flow records furnish the most reliable data for this purpose. It is unfortunately the case, however, that the establishment of stream-gauging stations must be limited to one or two points on the larger streams, on account of the first cost and the upkeep. There are many projects, however, both public and private, which depend on the supply from the tributaries of large streams, or from small streams. There are also many short stream-gauging records which do not extend over years of deficient precipitation, and, without proper corrections, are likely to be as misleading as precipitation records unsupported by stream-flow data. To fill these deficiencies, precipitation records judiciously used in conjunction with the available stream-flow records are of great value, and afford the only adequate information, obtainable at reasonable cost, with which to supplement insufficient stream-flow records. It is toward the filling of this gap, which every hydraulic engineer in the West at least recognizes as a serious one, that the Committee's efforts are directed.

* *Engineering News,* August 17th, 1916, p. 321.

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Third, the Committee's primary idea, in formulating its suggestions for improvement of the Weather Bureau Service, was not an increase in the volume of data collected, although that might be an incidental result, but an improvement in quality, to be obtained by a more studied choice of locations at which data are observed, and more intensive study and interpretation of the data thus obtained. For instance, in the matter of precipitation data, the hydraulic engineer wants to know: First, the average annual areal precipitation for all portions of a given drainage area, and the annual variations therefrom; and second, the monthly, daily, and hourly precipitation at one or more points in the area, depending on its size. As Mr. W. S. Post well suggests, this information can be obtained for Pacific Coast mountain drainage areas by determining the shape of the isohyetose lines from a few years records at a number of well-selected stations, after which one or two base stations would provide all the information needed. (This, of course, would not be sufficient for regions where a large proportion of the annual rainfall occurs in detached local storms covering limited areas.) By working out a progressive programme for a State along these lines, the total number of stations reporting at any one time could be kept constant, although the geographical location of stations would vary. As contrasted with such a programme, there is the present more or less haphazard distribution of rainfall stations, without regard to the ultimate usefulness of the record. The Committee recognizes the fact that to plan and execute such a programme would require much intelligent study and supervision by a practical and technically trained field man. The Committee wishes to emphasize again, however, that its efforts are actuated by a desire to see generally inaugurated a progressive, practical, co-operative, and broad handling of meteorological problems, to the end that these problems may be solved. Mechanically observing data for the sole purpose of tabulation and accumulation in the archives does not solve these problems, no matter how many stations are reporting.

The Committee wishes to correct an error which was inadvertently made in the paper. The number of co-operative stations in California reporting precipitation in 1913, as shown by the Annual Summary, is practically 300, instead of 192. Of these stations 8% are within 25 miles of the coast, 43% are in interior valleys and foot-hills, 7% are in the Great Basin and Desert, and 42% (instead of 29%) are in regions of broken topography where elevations exceed 2 000 ft. Of these, approximately 30 (instead of 21 as stated), were above 4 000 ft. Hence, out of the total, 10% can be considered as mountain stations. It should also be stated that subsequent to 1913 there have been published reports of snowfall data at from 40 to 50 stations in the Sierra Nevada Mountains. These data have been expressed in snow depths,

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however, and not in terms of equivalent water, so that they are of little use for water supply estimates.

The individual discussions will now be taken up in detail: Mr. Grover* suggests, as the reason that so few records have been obtained by the Weather Bureau in mountainous regions, the difficulty in devising self-regulating instruments. The Committee believes that, due to the rapid development which has occurred in the West during the past few years, there would be found many more reliable observers in mountain areas than might be supposed. This has been the experience of the members of the Committee. The Committee respectfully suggests that, if the local officers of the Weather Bureau were given more assistants and were allowed to become intimately familiar with the mountain portions of their districts, many good observers could be found, in areas from which records are at present lacking, by obtaining the co-operation of such local residents and by the establishment of stations in the isolated areas in charge of trained men. As the Committee has suggested, it should be entirely feasible to obtain the desired records within a reasonable cost without waiting for the development of automatically recording instruments. The Committee has already commented on Mr. Grover's reference to the possible misinterpretation of its original report, and wishes here to express its appreciation to Mr. Grover for having brought out this matter with such clearness.

Mr. Post's discussion† contains several very practical suggestions which are especially valuable, as they are drawn from his experience in making rainfall and run-off observations over a large area of mountain drainage. With regard to the publication of early records, of which, as Mr. Post states, many exist in California, it has been stated to the Committee, by the Chief of the Weather Bureau, that if a sufficient volume of such data be assembled and compiled by the Local Association, and presented to the Weather Bureau, the latter will undertake their publication in bulletin form under separate cover. The Committee is still engaged in gathering data of this character, and will appreciate any which members or others may be able to furnish.

Mr. Charles T. Leeds‡ brings out very clearly the diversity of work which has been assigned by law to the Weather Bureau, and the limitations under which it is working with respect to the needs of the water supply interests of the West. The force of his statements has been brought home to the Committee by its own investigations, and it fully agrees that the proper method of obtaining desired improvements is through Congressional action; and furthermore, that, "The

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* *Proceedings*, Am. Soc. C. E., for April, 1915, p. 927.

† *Proceedings*, Am. Soc. C. E., for May, 1915, p. 1219.

‡ *Proceedings*, Am. Soc. C. E., for August, 1915, p. 1587.

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Engineering Profession is probably in a better position to bring this to the attention of Congress than any other body of men." Mr. Leeds has done well to mention the many lines of special research work undertaken by the Weather Bureau. With regard to the evaporation studies made by that Bureau at Salton Sea about 6 years ago, however, attention is drawn to the fact that the data thus obtained have never been made available to the public. The Committee agrees with Mr. Leeds that stations for the collection and distribution of climatic information should be maintained in the centers of population. The Committee believes, however, that fully equipped (or regular) observation stations should be at a distance from buildings and other objects and influences which, in cities, cause abnormal conditions. The Committee suggests that both requirements could be met by establishing regular stations in Government-owned buildings and on Government-owned land, at the military and lighthouse reservations, of which many exist on the Pacific Coast, for example, the Presidio at San Francisco, Point Arguello near San Luis Obispo, Fort Rosecrans at San Diego, etc. There are military reservations adjoining practically all large coast centers of population, at which communication is fully as good as in the congested business district. The maintenance of such stations would, without doubt, be less than that of the expensive office quarters usually occupied by the Weather Bureau, and, in addition, the abnormal conditions existing on top of office buildings would be eliminated, without sacrificing the requirement of easy communication.

The Committee has not been unaware of the course traveled by the typical storm centers of this continent, as suggested by Mr. Leeds, but believes that, by reason of the large diameter of these storm centers, atmospheric changes occur no later on the west slopes of the mountain ranges of the Pacific Coast than they do on the Coast itself, and that records obtained from judiciously selected mountain stations would be of just as great value in forecasting weather conditions. In this matter, however, the Committee heartily agrees with Mr. Tibbetts.* that

"Two observation stations in the ocean would be of more value, in predicting several days in advance the occurrence of heavy storms, than all the present stations in California combined."

The Committee is indebted to Mr. Tibbetts for his valuable and instructive suggestions, as they represent the experience and viewpoint of an engineer familiar with conditions in Central and Northern California.

In connection with Mr. Tibbett's suggestion, it is proper to call attention to the fact that, in the course of an oral discussion of the matter at a meeting of the Southern California Association of Mem-

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* *Proceedings*, Am. Soc. C. E., for March, 1916, p. 382.

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bers of the American Society of Civil Engineers, William Mulholland, M. Am. Soc. C. E., made the suggestion that naval vessels be used for the purpose of observing and reporting by wireless weather conditions off the coast. Though it seems to the Committee that the primary objects of the naval establishment would probably interfere with regular service of this character, such information as might be given, incident to the position of the vessel, would certainly be of high value. A much broader and more comprehensive service could be obtained very readily, however, by establishing the practice of communicating, through the Government wireless stations, with all shipping within range of the station, whether it be naval, coastwise, or deep-sea. The simple request of the American Consul at a foreign port would be quite sufficient, in a vast majority of cases, to insure the prompt attention of shipmasters to wireless inquiries from our Government stations, as to their position, direction of wind, barometric pressure, temperature, etc., and, with such a custom of communication established, the reciprocal advantage to the shipmasters would be correspondingly great. The utilization of such a source of meteorological data would involve practically no expense, would call for no additional plant or labor, and, the Committee believes, would result in a very great improvement in the advance with which conditions could be forecast, and in forecasts of far greater reliability.

Mr. Lippincott has well expressed the administrative policy of the Weather Bureau with respect to the West. The visit of the present Chief of the Weather Bureau to the Pacific Coast in 1915, and the recent extension of the mountain climatological work of that Bureau in Southern California, would indicate, however, that an effort is being made by the present administration to bring about a change for the better. The Committee can agree with Mr. Lippincott that the reason so few field inspection trips are made, by local officers of the Weather Bureau, is because of the "system". Before a local officer can leave his station for official business of any kind, no matter how trivial, he must go through a long process of correspondence with the Washington office. This sometimes consumes a month, and even then the authorization is frequently curtailed, and is often denied. There is small wonder, therefore, that co-operative records are often broken or discontinued. Such stations should be visited at least once a year, and at no pre-arranged time, in order to insure accurate results and the active co-operation of the observer.

As a result of the discussions and the Committee's further investigation of the subject, it is believed that the following general conclusions are warranted:

1.—That the law creating the United States Weather Bureau prescribes a wide range of duties, among which the gathering of precipitation records is but incidental.

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2.—That there is an urgent demand among engineers, throughout the United States, for more complete precipitation data throughout mountain drainage areas, to be used in conjunction with stream-flow data.

3.—That the present fiscal regulations, organization, and administrative policy of the Weather Bureau are not adapted to the task of gathering complete precipitation data of the character desired by engineers.

4.—That Congressional action should be sought, either to change the organization of the Weather Bureau, so that the desired result can be accomplished, or else that the duty of observing all factors affecting stream flow be turned over to the Water Resources Branch of the United States Geological Survey, with the appropriation of sufficient additional funds to carry on the work efficiently.

The Committee is continuing its studies in this matter, and its conclusions and suggestions, as just stated, are not to be considered final. The question of the establishment of a Bureau of Public Works, which shall co-ordinate all engineering activities of the Federal Government, and prevent the present duplication of effort, is believed worthy of careful study. Any further suggestions from the membership of the Society bearing on these matters will be greatly appreciated by the Committee.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852


## PAPERS AND DISCUSSIONS



# A COMPLETE METHOD FOR THE CLASSIFICATION OF IRRIGABLE LANDS

## Discussion.*

BY G. N. HOUSTON, M. AM. SOC. C. E.

G. N. HOUSTON,† M. AM. SOC. C. E. (by letter).‡—Irrigation development comes under one of two classes: that affecting areas where irrigation is a necessity, and that affecting areas where it is only desirable. Irrigation is a necessity where, other conditions being favorable, the precipitation is insufficient to produce a profitable crop. It is desirable where the precipitation, other conditions being favorable, is insufficient to produce the maximum profitable yield of the particular crops adapted to the region. Mr. Houston.

The possibilities under the first class have been about exhausted, except as the area is extended by conserving the water and rendering it more efficient in crop production. Under the second class, there is a great deal yet to be accomplished.

It has not been considered necessary to make any extensive study of the climatic conditions in connection with irrigation development in the United States, because the projects have been nearly all confined to the arid and semi-arid districts, where the desirability of irrigation had been demonstrated in a small way by the individual farmer, or by the small community of settlers. It was assumed that like benefits would accrue, in this district, wherever water could be applied to arable land, and the engineer has been concerned principally with transporting and applying this water.

In connection with projects of the second class, however, that is, where irrigation is not a necessity but is simply desirable for increasing

* Discussion of the paper by F. H. Peters, Assoc. M. Am. Soc. C. E., continued from November, 1916, *Proceedings*.

† Denver, Colo.

‡ Received by the Secretary, January 6th, 1917.

Mr. Houston.

the yield of an already profitable crop, the development is reaching out into the more or less humid regions, and the question of desirability is becoming more and more complicated. In these cases a study of the climatic conditions is necessary, especially with regard to the precipitation, in order to determine whether the probable additional profits due to irrigation would be sufficient to justify the cost of the system.

Monthly averages of the precipitation as published do not furnish the necessary data for this study. The daily distribution of the rainfall, or, in other words, the occurrence of drought periods during the cropping season, must be considered, as this is the chief factor affecting the yield of the crop. In connection with the work of which the paper treats, the writer had occasion to make such a study of the climatic conditions of several points in Southern Alberta and Northern Montana, a summary of which is given in Table 5. A graph was made showing the daily rainfall during 11 years for each of the locations, and the drought periods were picked out. Column 1 shows the length of each period; Column 2 shows the total precipitation which occurred during each drought; Column 3 shows the number of days during the drought on which rain fell; and Column 4 shows the maximum rainfall on any one day during the drought.

The effect which a period of light rainfall may have on a crop depends on a number of factors, such as the rate of evaporation, as affected by humidity and wind velocity, the kind of crop and the stage of its growth, the general texture of the soil, and whether or not it has had any preparation to receive and conserve the moisture. These matters must be given due weight in determining the desirability of irrigation especially on those tracts where the climatic conditions are nearly humid.

In the development of irrigation projects, too little attention has been paid to the character of the soil of the land proposed to be put under water. This, however, is simply one phase of the general tendency of the promoter, the investor, and in many cases the engineer, to disregard the conditions under which the investment is to be returned. Large expenditures have been made in order to deliver water to land the character of which, except as regards surface appearance, has been practically unknown.

Some of this land is impregnated so heavily with alkaline salts that it cannot produce sufficient under irrigation to return interest on the investment. Where the alkali shows on the surface in large white patches, there is little danger of the land being included in an irrigation scheme without provision being made for its reclamation. The subsoil, however, may be very heavily impregnated and there may be very slight indications, or none at all, on the surface, until after the project is in operation, when the salts are brought to the surface

TABLE 5.—SUMMARY OF THE STUDIES OF DROUGHT CONDITIONS IN SOUTHERN ALBERTA AND NORTHERN MONTANA.

| Year. | GLEICHEN, ALBERTA. | | | | MEDICINE HAT, ALBERTA. | | | | LETHBRIDGE, ALBERTA. | | | | BOSEMAN, MONT. | | | | CHINOOK, MONT. | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | (1) Length of period, in days. | (2) Total precipitation in period. | (3) Number of days on which rain fell. | (4) Maximum for one day. | (1) Length of period, in days. | (2) Total precipitation in period. | (3) Number of days on which rain fell. | (4) Maximum for one day. | (1) Length of period, in days. | (2) Total precipitation in period. | (3) Number of days on which rain fell. | (4) Maximum for one day. | (1) Length of period, in days. | (2) Total precipitation in period. | (3) Number of days on which rain fell. | (4) Maximum for one day. | (1) Length of period, in days. | (2) Total precipitation in period. | (3) Number of days on which rain fell. | (4) Maximum for one day. |
| 1913 | 27 | 0.00 | 0 | 0.00 | 27 | 0.17 | 4 | 0.10 | 31 | 0.53 | 6 | 0.29 | 22 | 0.33 | 8 | 0.17 | 25 | 0.41 | 2 | 0.25 |
| | 23 | 0.02 | 1 | 0.02 | 21 | 0.23 | 2 | 0.22 | 19 | 0.27 | 4 | 0.20 | 21 | 0.59 | 4 | 0.25 | 17 | 0.18 | 1 | 0.18 |
| | 17 | 0.33 | 2 | 0.25 | | | | | | | | | 17 | 0.27 | 3 | 0.11 | | | | |
| 1912 | 18 | 0.17 | 3 | 0.10 | 26 | 0.39 | 7 | 0.10 | 20 | 0.34 | 6 | 0.16 | 24 | 0.68 | 7 | 0.19 | 20 | 0.40 | 2 | 0.38 |
| | 25 | 0.34 | 2 | 0.26 | 17 | 0.17 | 3 | 0.13 | 23 | 0.07 | 2 | 0.06 | 23 | 0.64 | 6 | 0.20 | 39 | 0.48 | 3 | 0.22 |
| | 17 | 0.20 | 2 | 0.15 | 19 | 0.30 | 3 | 0.15 | | | | | | | | | 17 | 0.23 | 2 | 0.19 |
| | 20 | 0.26 | 3 | 0.10 | 21 | 0.33 | 6 | 0.17 | | | | | | | | | | | | |
| 1911 | 24 | 0.00 | 0 | 0.00 | 20 | 0.30 | 8 | 0.10 | 18 | 0.13 | 3 | 0.06 | 22 | 0.85 | 5 | 0.41 | 20 | 0.12 | 1 | 0.12 |
| | 17 | 0.00 | 0 | 0.00 | | | | | 24 | 0.48 | 8 | 0.14 | 24 | 0.28 | 3 | 0.15 | | | | |
| 1910 | 31 | 0.55 | 2 | 0.30 | 63 | 0.86 | 11 | 0.16 | 41 | 0.45 | 8 | 0.16 | 69 | 1.02 | 18 | 0.25 | 31 | 0.52 | 7 | 0.12 |
| | 64 | 0.00 | 0 | 0.00 | 31 | 0.53 | 5 | 0.22 | 39 | 0.08 | 2 | 0.06 | | | | | 37 | 0.08 | 3 | 0.03 |
| 1909 | 27 | 0.56 | 4 | 0.44 | 25 | 0.49 | 2 | 0.39 | 43 | 0.80 | 11 | 0.22 | 26 | 0.71 | 8 | 0.19 | 14 | 0.03 | 1 | 0.03 |
| | | | | | | | | | 19 | 0.15 | 4 | 0.05 | 14 | 0.12 | 2 | 0.09 | 27 | 0.50 | 2 | 0.33 |
| 1908 | 36 | 0.55 | 4 | 0.25 | 25 | 0.24 | 4 | 0.07 | 49 | 0.54 | 5 | 0.19 | 57 | 1.05 | 12 | 0.25 | 19 | 0.34 | 3 | 0.19 |
| | | | | | 32 | 0.44 | 7 | 0.17 | | | | | | | | | 43 | 0.19 | 2 | 0.11 |
| | | | | | 24 | 0.40 | 5 | 0.14 | | | | | | | | | 18 | 0.15 | 3 | 0.08 |
| 1907 | 19 | 0.14 | 1 | 0.14 | 19 | 0.05 | 1 | 0.05 | 21 | 0.43 | 7 | 0.03 | 17 | 0.56 | 6 | 0.30 | 19 | 0.07 | 1 | 0.07 |
| | 21 | 0.32 | 2 | 0.18 | 17 | 0.20 | 2 | 0.15 | | | | | | | | | 28 | 0.20 | 4 | 0.10 |
| | | | | | 20 | 0.23 | 5 | 0.10 | | | | | | | | | | | | |
| | | | | | 24 | 0.30 | 3 | 0.14 | | | | | | | | | | | | |
| 1906 | 19 | 0.30 | 3 | 0.13 | 13 | 0.00 | 0 | 0.00 | 31 | 0.61 | 4 | 0.21 | 38 | 0.50 | 6 | 0.23 | 43 | 0.09 | 1 | 0.09 |
| | 28 | 0.39 | 1 | 0.39 | 49 | 0.40 | 6 | 0.10 | 16 | 0.13 | 1 | 0.13 | | | | | | | | |
| 1905 | 20 | 0.23 | 3 | 0.10 | 17 | 0.34 | 4 | 0.12 | 21 | 0.36 | 6 | 0.25 | 46 | 0.975 | 10 | 0.30 | 18 | 0.22 | 1 | 0.22 |
| | 19 | 0.32 | 3 | 0.22 | 20 | 0.14 | 2 | 0.08 | 25 | 0.24 | 5 | 0.08 | | | | | 14 | 0.00 | 0 | 0.00 |
| | | | | | 32 | 0.64 | 7 | 0.16 | | | | | | | | | | | | |
| 1904 | 21 | 0.25 | 2 | 0.20 | 19 | 0.38 | 4 | 0.14 | 46 | 0.42 | 4 | 0.19 | 20 | 0.48 | 7 | 0.12 | 18 | 0.10 | 1 | 0.10 |
| | 45 | 0.39 | 4 | 0.13 | 43 | 0.84 | 7 | 0.20 | | | | | 17 | 0.06 | 2 | 0.05 | 21 | 0.08 | 1 | 0.08 |
| | | | | | 22 | 0.46 | 2 | 0.36 | | | | | 31 | 0.95 | 8 | 0.38 | | | | |
| 1903 | 14 | 0.25 | 1 | 0.25 | 65 | 1.06 | 12 | 0.35 | 16 | 0.09 | 2 | 0.08 | 29 | 0.50 | 7 | 0.23 | 26 | 0.47 | 3 | 0.30 |
| | 31 | 0.54 | 5 | 0.24 | | | | | 25 | 0.28 | 6 | 0.07 | | | | | 18 | 0.15 | 1 | 0.15 |
| | 17 | 0.12 | 3 | 0.08 | | | | | 21 | 0.29 | 3 | 0.19 | | | | | | | | |
| Mean.... | 25 | 0.260 | 2.1 | 0.164 | 27.3 | 0.380 | 4.69 | 0.156 | 27.4 | 0.334 | 4.8 | 0.144 | 28.7 | 0.587 | 6.5 | 0.21 | 22.4 | 0.222 | 2.0 | 0.149 |

Mr. Houston.

| Group No. 164 | | |
|---|---|---|
| Sec. | T. | R. |
| 7 | 23 | 26 |
| Cairnhill Dist. | | |

SOIL INVESTIGATION FIELD NOTES

June 11, 1914

(a) Apparatus used in collecting. post-hole digger no 6

(b) Approx. distance from Stream, Lake, or other water; about ½ mile S. of slough and ¼ mile S. of ditch

(c) Approx. elevation above or below water; 20' below ditch and 10' above slough

(d) Land—~~In Crop~~ ~~Summerfallow~~—Unbroken ~~Formerly cropped but idle.~~

(e) Present Crop None ~~Exceptionally good, Good, Average, Poor.~~

(f) If unbroken, character of native vegetation; prairie grass.

(g) ~~Irrigated~~ Unirrigated

(h) If irrigated, the date of last application of water

(i) Date of last rainfall on the area: June 9th, 1914.

(j) Are conditions where sample is taken average for the ¼ section? yes, for west half of land below ditch

(k) Does white alkali appear on the surface? yes, slight indications.

(l) Has it affected the vegetation? yes; a few small bare spots.

(m) Remarks; with thin grass around the edges. otherwise, a good crop of prairie grass on this area.

| Sample | Depth: from | Depth: to | Moisture | Color. | Description |
|---|---|---|---|---|---|
| A 164 | 0.0 | 0.4 | M. | dark gray. | clay and loam. |
| B 164 | 0.4 | 2.0 | D. | gray. | clay loam. |
| C 164 | 2.0 | 3.5 | D &. | gray-brown | clay loam. |
| D 164 | 3.5 | 6.0 | W &. | yellow. | sand. |
| E --- | -- | -- | --- | ------- | ------- |
| F --- | -- | -- | --- | ------- | ------- |

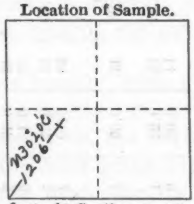
(n) Depth of the surface soil; 0.4 ft.

(o) Depths to which roots penetrate; 5.0 ft

(p) Depth to the water table; 5.5 ft

Remarks: hole fell in at the bottom, apparently quick sand.

Location of Sample.




S. W. ¼ Section 7-23-26

| Collected by: | Approved: |
|---|---|
| W.E.G.H. | GWH |
| Asst. Field Eng. | Eng. in Charge |

FIG. 8.

by the direct application of water or seepage from a near-by ditch. The result is that the investor suddenly realizes that he will be obliged to build an expensive drainage system, to wash out the alkali, or cut out this land entirely from the project. In the latter case, in order that the investment shall be returned, the remaining land must be burdened with an additional tax, in proportion to the part excluded, which possibly it is not able to stand. Mr. Houston.

No irrigation project should be built until a careful study of the soil conditions has been made, not only in regard to the topography and alkaline content, but also a classification as to its general texture and fertility. With data of this kind as a basis, the prices of the various classes of land can be fixed, and a variable system of payments arranged so that there will be a reasonable possibility of the settler meeting his obligation from the produce of the land.

In the case under discussion, a typical sample of the field notes of the soil investigations is shown in Fig. 8.

A post-hole digger was found to be most satisfactory for obtaining the samples, although several kinds of soil augers were tried. The depth of the hole was usually 6 ft. Samples were taken whenever the soil changed in color or texture. These were packed in 1-qt. glass preserve jars, in order to retain their moisture content, and shipped to the Dominion Chemist for analysis.

The maps were carried in the field in a galvanized-iron case, hinged at the right side and having a beaver-board back. The following abbreviations were used in the soil investigation field notes.

| | | |
|---|---|---|
| *W &.* | Very wet. | Water in the hole. |
| *W.* | Wet. | Water will drip from sample. |
| *D &.* | Very damp. | Water can be squeezed from sample. |
| *D.* | Damp. | Slightly soggy. |
| *D—.* | Less damp. | Putty-like. |
| *M.* | Average soil mixture. | |
| *P.* | Dry and powdery. | |

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## THE YALE BOWL

### Discussion.*

By Henry C. Hitt, Assoc. M. Am. Soc. C. E.

Henry C. Hitt,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer has been very much interested in this particularly well written description of an unusual piece of construction. The author has given a very complete and satisfactory description, full of the data one wants, of the details of the design and of the construction methods. Mr. Hitt.

The design certainly appears to be very successful. The "bowl" shape is an outdoor adaptation of old principles of auditorium design, and undoubtedly provides the most ideal seating scheme, as well as being architecturally beautiful. It is strange that it has not been used more frequently with other stadia. The slate-colored concrete, with a trim of the inner retaining wall and the tunnel portals in natural color, the comfortable seats, the beautiful elliptical shape, and the careful orientation as to the afternoon sun, must all go to make the structure a credit to every one responsible.

The author, in comparing the Bowl with other modern stadia, states that it is probably the cheapest per sitting unit. In this, however, he is in error, as the Tacoma Stadium, which the Bowl resembles in some ways, has cost, completely equipped, only about 85% of what the Bowl, in its unfinished condition, with a portion of the seats temporary and the dressing rooms not built, is reported by the author to have cost, per unit of seating capacity.

* Discussion of the paper by Charles A. Ferry, M. Am. Soc. C. E., continued from December, 1916, *Proceedings*.

† Seattle, Wash.

‡ Received by the Secretary, January 5th, 1917.

Mr. Hitt. Thus far, the total cost, of every kind, of the Tacoma structure, built in 1910, has been $147 000, and an estimate of the seating capacity, based on the same assumptions as used by the author, is 23 784, giving a unit cost of $6.20. The architects claim a seating capacity of 32 000, and more than 35 000 people have actually occupied the stadium at one time, with none of them on the field, but part of them standing.

The site has also a reserve capacity of about 15 000, as the sodded terraces rise for 43 ft. above the top of the seats around more than half the perimeter, at a proper slope for future seating as the needs of the rapidly growing city demand it. The addition of these seats would materially cut down the unit cost of the whole.

The two structures were designed to fill widely different conditions, but are similar in the result. The fields are of practically the same area, but where the Yale Bowl was designed strictly for football and seating capacity, with few or no limiting conditions, the site of the Tacoma Stadium was limited, and the field had to be adapted for the general use of the people and the adjoining High School, for track work, football, baseball, pageants, concerts, or political meetings. This made a complicated problem, which was most happily solved.

The seating scheme is similar to one end only of the Yale structure, with every occupant able to see the whole field, and most, if not all, the audience. At Tacoma, however, the designers had a natural location, of which they took the fullest advantage. The site is on a promontory in the very heart of the city, with the playing field 140 ft. above the harbor. From the Stadium seats one gets a view probably not equalled by that from any similar structure. Spread before the spectator is a magnificent sweep of busy Tacoma Harbor, green hills, and snow-clad mountains. The Stadium is flanked on each side by beautiful buildings rising above the seats, on one side the Stadium High School and on the other the State Historical Museum.

Although the site was naturally somewhat adapted to the purpose, there was a very large quantity of excavation, more per unit of seating capacity than with the Yale Bowl, and about two-thirds of it proved to be very hard cemented gravel. All the seats are carried on columns and radial girders about 20 ft. apart, the risers of the seats, 3½ in. thick, reinforced with a structural frame carrying the seat load between the girders. Part of the columns, at the outer end, are carried on piling, and part of the field at this end is a fill of 147 ft., but there has been no trouble from settlement.

Twenty-eight arc-light projectors are placed around the field above the seats, and twelve more are suspended by cables, high above the field, affording ample light for evening performances. The acoustics are very fine, and concerts have been entirely successful. The stadium is fully equipped with dressing rooms, and comfort stations for the spectators.

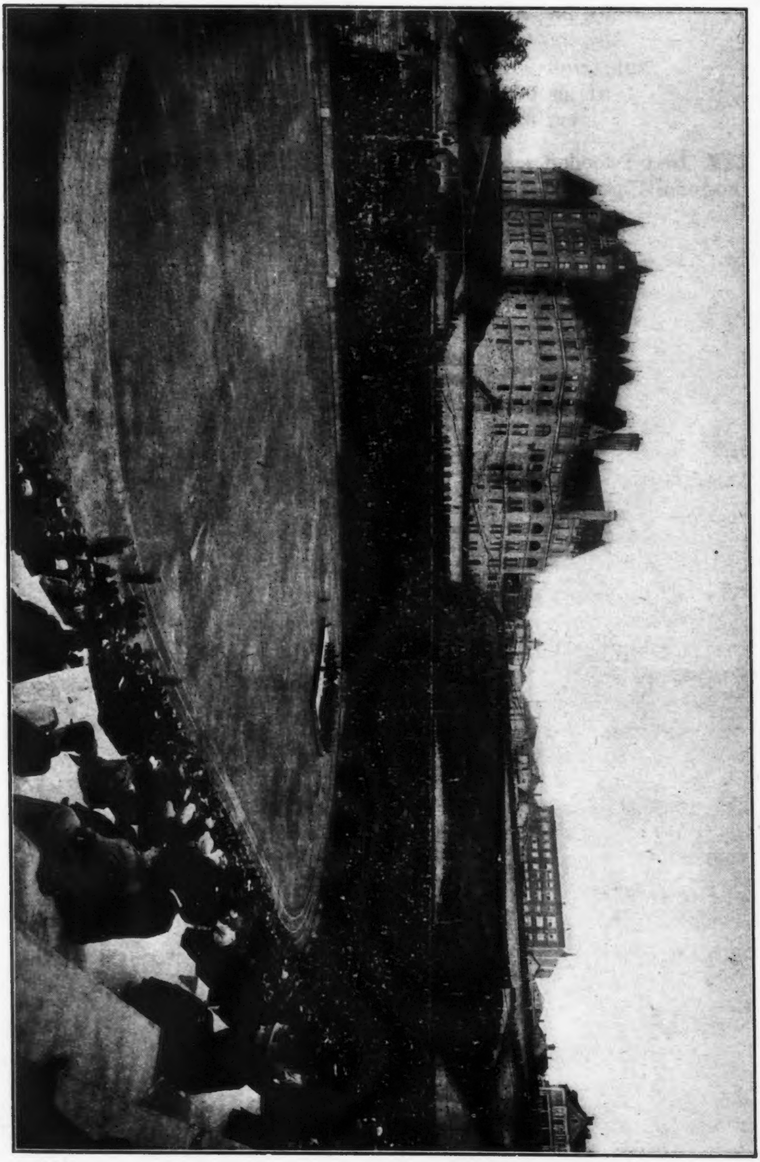


FIG. 1.—THE TACOMA STADIUM.

The principal quantities were, approximately: Mr. Hitt.

| | |
|---|---|
| Earthwork .................... | 185 000 cu. yd. |
| Reinforced concrete............ | 10 000 cu. yd. |
| Reinforcing steel............... | 160 tons, plus |
| Clinton fabric.................. | 130 000 sq. ft. |
| Area of field................... | 3.6 acres |

The Tacoma Stadium was built by the Tacoma School Board, Mr. Frederick Heath having been the architect, and Mr. L. A. Nicholson the engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## THE VALUATION OF LAND
### Discussion.*

By J. S. Walker, Esq.

**J. S. Walker,† Esq.—The speaker, until lately, has been connected with the Tax Department of Jersey City, and in his position witnessed many contests between the City and the corporations. In connection therewith he has heard able men discuss the various phases of this question. He realizes the fact that this paper represents a great deal of study, is to the point, and is correct.** Mr. Walker.

**The most interesting feature is the theory of the value of the fractional part of a lot. As Mr. Kelly has stated, Hoffman's opinion, in 1870, was that the front half of a lot was worth two-thirds of its total value. Hoffman produces a series of numbers; as to how he derived them we are not told, but they happen to be a series representing a conic section.**

**The conversion of Mr. Bernard's 150-ft. rule into the 100-ft. rule by Mr. Jerrard, agrees with the Hoffman Rule, which the speaker thinks is very peculiar.**

**The little investigating the speaker has done tends to show that the problem of the relation of depth to value results in an equation of the second degree, between two variables, and is an equation of a conic section.**

**In practice a curve is constructed or adopted, and, under certain conditions if it gives too much or not enough value, some other curve is copied in whole or in part. The curve then varies in such a manner that it cannot be represented by a formula. Such being the case, no two conditions are treated according to the same law.**

* Discussion of the paper by L. P. Jerrard, Jun. Am. Soc. C. E., continued from January, 1917, *Proceedings*.

† Jersey City, N. J.

Mr. Walker. A case in point is that of a district in transition from one use to another, such as apartments taking the place of private dwellings, the appraiser claiming that more value is found to a greater depth for land on which was built apartments than that devoted to private dwellings, and a variation of the curve in use was applied.

In 1912, or about that time, Newark had curves which differed from those in use at present, to which the foregoing objections would apply. The subject calls for extended and exact study.

The Manufacturers Appraisal Company, of Cleveland, Ohio, which uses the "Somers Unit System of Realty Valuation", does not reveal the method of calculating the value of corner lots, irregular lots, or reflected influences. It appears that a city hires only the use of the system, and, should it be desired to re-adjust values, it would be necessary to re-hire the system. Nevertheless, all things about the Somers System are not kept secret, and it has may good features.

First of all, it is a system that is carefully worked out. The speaker thinks it overcomes some of the objections referred to by one of the speakers.

A booklet, published by the City of Cleveland, Ohio, entitled "The First Quadrennial Assessment, Cleveland, Ohio", shows how Cleveland was assessed by the Somers System in 1910, and gives many details and charts.

Mr. W. I. King,* appears to have gotten nearer to the heart of the system than any one else, and shows some of the Somers charts not found elsewhere, but he says:

"Since the scientific accuracy of the Somers tables is not proven, and since these tables and their derivation are shrouded in secrecy, it seems necessary to derive a method of corner valuation, and ultimately a series of charts, which may lay some claim to a scientific basis, and at the same time, accord with actual conditions."

This bulletin is of value to those interested in taxation.

In assessment work, Mr. Somers lays much stress on the value of community opinion. He writes as follows with reference to an assessment in Des Moines, Iowa:†

"The first public meeting was held; * * * about seventy-five prominent citizens were present, including a majority of the largest property owners. After the session a committee of citizens and property owners was selected by the Mayor and Assessor to co-operate with, and assess the latter.

"At the outset it was decided to avoid discussion of actual values at first and to deal with relative values only. Therefore the efforts

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* *Bulletin No. 689* of the University of Wisconsin entitled "The Assessment of Urban Property."

† As published in the June, 1913, issue of the *Somers System News*.

of the Committee were devoted to expressing in percentage the relative values of a Somers unit foot on each side of every block. Mr. Walker.

"It was the unanimous opinion that the most valuable frontage in the city was on the north side of West Walnut Street between Sixth and Seventh Streets. This the Committee called $x$, or 100 per cent. Then, at the several meetings, the relative value of a unit foot in every block throughout the West Side was ascertained.

"When the relative values were compiled, a map of the district was prepared, and the percentages placed thereon. Copies of this map were distributed and printed in the newspapers; another meeting was called, and criticism of the relative values invited."

Another section of the city was likewise treated.

"When the two committees had completed their labors, it remained only for the assessor to determine $x$, or the 100%, on West Des Moines Street." The value of $x$, or 100%, also came under discussion. "Calculating clerks were then employed and trained in the methods of the Somers System. As soon as the unit values were definitely ascertained, the work of calculating began."

The opinion of the taxing authorities throughout the United States seems to be that the "Newark System" is fundamentally the best, but it has taken 20 years to build it up. In Jersey City, a system is being constructed by testing any idea that looks good in any system and is likely to develop into one which will be the best in the country.

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# PAPERS AND DISCUSSIONS



## TESTS OF CONCRETE SPECIMENS IN SEA WATER, AT BOSTON NAVY YARD

### Discussion.*

BY MESSRS. T. KENNARD THOMSON, J. J. YATES, J. R. MCCLINTOCK, S. B. WILLIAMSON, WALDO C. BRIGGS, CHARLES S. BILYEU, W. E. DAY, ROBERT RIDGWAY, AND GEORGE W. FULLER.

T. KENNARD THOMSON,† M. AM. SOC. C. E.—This is an interesting paper on an interesting subject, and should bring out a full discussion from experience in actual work. Mr. Thomson.

The author calls attention to the fact that a mixture of one part of cement to two parts of sand gave a better test than the mixtures which were not so rich in cement. This is undoubtedly due to the fact that a 1:2 mixture comes nearer to having the voids of the sand filled with cement than a poorer mixture, and, of course, a concrete with all the voids filled has a better chance of resisting the effects of frost and of any injurious chemical which may be in the water or air; and probably explains why the greatest damage to concrete is generally between high and low water.

Some years ago the speaker built a breakwater, off New Rochelle, which is subjected to storms with the full sweep of Long Island Sound behind them. A rock fill was used, up to the low-water mark, the depth of water at low water being 25 ft. at the outer end.

From low water upward the stones were carefully placed to a height of 14 ft. (except for the outer 50 ft.), on top of which a 12-in. concrete coping was placed, as individual stones would have been displaced by the storms, although the top of the breakwater was 9 ft. above ordinary high tide.

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* This discussion (of the paper by R. E. Bakenhus, M. Am. Soc. C. E., published in December, 1916, *Proceedings*, and presented at the meeting of January 3d, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Thomson.

At the outer end for 50 ft. the built-up stonework was only 8 ft. above low water, on top of which was placed a concrete block, 7 ft. high, 10 ft. wide on the base, 5 ft. on the top, and 50 ft. long; a small quantity of reinforcement was used.

This breakwater has stood successfully the severe storms of the Sound for some 11 years; but this year it has been noticed that a small quantity of the concrete had seriously deteriorated for a foot or so at the bottom and for about a foot in depth.

It was a little surprising, however, that this destruction occurred on the inside or sheltered portion of the breakwater, instead of outside, where it was exposed to the storms. This may have been due to the fact that on the exposed side of the breakwater any injurious chemicals from sewage, etc., would be washed away instead of remaining in place, as they would on the sheltered side.

It has been found impossible to get a concrete to withstand the action of impure, hot water, in some power-plants. In one case the efforts have been abandoned, and a wooden lining has been substituted for the concrete.

It would seem that the three principal causes for disintegration of ordinarily good concrete in sea water are: first, too coarse a mixture, in which the voids of the sand or stone are not filled—thus allowing chemicals to accumulate; second, alternate freezing and thawing, which do far more damage when the voids are not filled; and third, the action of injurious chemicals assisted by heat.

The remarks in the discussion about sand recall an attempt of a railroad contractor, many years ago, to make the railroad haul free of charge for 110 miles a fine beach sand, because the local sand appeared to contain too much loam. The speaker made three sets of tests: first, with regular testing quartz; second, with the loamy sand; and third, with the fine beach sand. The tensile strength developed by the briquettes of the quartz and loamy sand were almost the same, and those made with the clean but fine sand gave only half the strength of the others. As strength in the concrete was what was wanted, the work was finished with the loamy sand.

Mr. Yates.

J. J. YATES,* ASSOC. M. AM. SOC. C. E.—The question of the action of sea water on concrete is of great importance, as it affects the permanency of such a large number of important structures which have been, or may be, built in sea water. This paper adds much valuable information on a subject that has received far too little study.

The speaker, however, thinks that one important phase which has not been considered is injury to concrete structures by floating ice or floating débris, resulting in the breaking up of the surfaces and the exposure of the interior concrete to the action of sea water.

* New York City.

From observations of existing structures, extending over a period of about 15 years, the speaker is led to the opinion that, in setting, a surface is formed on concrete which will resist successfully the action of sea water on the cement; and that, as long as this surface is maintained in good condition, no destructive action due to sea water may be expected. In other words, he is of the opinion that concrete surfaces are first injured or broken up by drifting material, and that, after they are thus broken up, the concrete is not able to resist the action of the sea water. Observations indicate that this action, after it once starts, continues, and is accelerated by the wearing away of the disintegrated surfaces by drifting ice and other material, or by the alternate freezing and thawing of the concrete surfaces between tides. By one of these means, the disintegrated concrete is removed, continually exposing a fresh surface to the action of the sea water. Mr. Yates.

As examples of the action of sea water on concrete, the following are given:

The concrete masonry of the old center piers of the Hackensack and Passaic River Bridges of The Central Railroad Company of New Jersey were constructed about 1870 of Rosendale cement. The concrete of these piers was deposited in a circular, wrought-iron, floating caisson, which was sunk in position to a depth of about 12 ft. on a previously prepared pile foundation. About 1888 or 1889 the iron superstructure was renewed, making use of the old piers. When inspected about 1902 it was found that the iron caisson had completely disappeared, to a depth of several feet below low water, and that the concrete had been eaten away between high and low water to a depth, in some cases, of about 2 ft. These piers were repaired by patching with concrete behind a steel plate band extending around each pier.

The piers were not exposed to drifting material, as they were protected or practically enclosed by pile and timber fenders of a type generally used at draw-bridges. Considerable trouble was experienced, however, from ice forming around the piers. In fact, during extreme high tide conditions, the operation of the lower of the draw-bridges was seriously interfered with by ice clogging the bearing wheels on the center pier.

In 1912 both the substructure and the superstructure were removed, and temporary structures were built at one side for the purpose of building new bridges.

During 1903 to 1905 The Central Railroad Company of New Jersey replaced its draw-bridge over Newark Bay, about $\frac{3}{4}$ mile from its mouth or junction with Staten Island Sound, with two bascule spans. The seven concrete piers required were deposited in air within wooden

Mr. Yates. floating caissons which were sunk to a depth of about 30 ft. on prepared pile foundations. Sea water was not allowed to come in contact with the concrete until it had had time to become thoroughly set.

The concrete was composed of 1 part of cement, 3 parts of sand, and 5 parts of 1½-in. broken trap rock. The following is a representative test analysis of the cement used:

| | |
|---|---|
| Sulphuric acid........1.38% | Magnesia ............1.54% |
| Gravity .............3.06 | Water and steam tests.O. K. |
| Initial set............1 hr. | Hard set.......4 hr. 30 min. |
| Residue, No. 200 sieve.27.5% | Residue, No. 100 sieve..8.0% |

One-day test................385 lb.
One-week test..............716 lb.
One-week test, sand.........315 lb.

The concrete was deposited under the supervision of experienced men, who stated that all the piers were treated in the same manner.

Shortly after the timber caisson was removed, Pier 5 was injured between high and low water by a floating barge. This injury was repaired by patching with concrete. Examinations of this pier in 1909 showed that the patch had disappeared entirely, and that the concrete was being eaten away gradually between high and low water at the point of injury. All the remaining piers except two showed some deterioration of the concrete between high and low water. In the case of Pier 5 the concrete was eaten away to a depth of 20 in. Two piers showed practically no deterioration. An examination by a diver in 1909 showed that there was no deterioration of any of the piers below a point about 1 ft. below mean low water. From observations, the piers most affected were those which had the greatest exposure to floating ice or drift.

Attempts were made to get samples of the disintegrated concrete for the purpose of making a chemical analysis to determine, if practicable, the action of the sea water. This was found impracticable, but when the samples were first removed and still wet, a large number of crystals of a white salt were visible, and these seemed to disappear entirely as the concrete dried out.

These piers have been repaired several times—once with a cement gun, which latter repair lasted about 4 years. The repairs have somewhat retarded the disintegration of the concrete, but the destructive action has been found to be practically continuous. It is very noticeable that the action is not general over the whole surface, but is rather confined to spots, such as the injured section of Pier 5.

Another example may be cited, not particularly for the disintegrating action of the sea water, but rather on account of the fact that a sample of the white salt, mentioned in reference to the Newark Bay

Bridge, was obtained and analyzed; this is the bridge of The Central Railroad Company of New Jersey over the Shrewsbury River about a mile from its outlet in Sandy Hook Bay, and built in 1913. Ten piers were constructed in sea water, and of these, two showed signs of failure about a year after they were built. These piers were rebuilt, and an investigation into the cause of the failure developed the fact that, in both of them, the concrete of the foundations, which had been deposited in water with a tremie, had been deposited at low tide, when the chute to the tremie was very flat. The concrete mixer had been on a floating barge, and the low tide had caused the flat condition of the chute which resulted in the concrete being mixed very wet and the tremie not being kept full of concrete during the process of depositing. Mr. Yates.

The concrete foundations of these piers failed completely, and when removed showed stratification, consisting of alternate layers of a putty-like substance and of sand and gravel with a little cement, or rather, a very weak mixture of concrete. Samples of this mixture were taken, and the small crystals of white salt were plainly visible when the concrete was first removed and before it had time to dry out. Enough of this white powder was collected, and an analysis was made by Mr. C. M. Chapman, Engineer of Tests, of Westinghouse, Church, Kerr and Company, a copy of which is as follows:

"We have made an analysis of the sample of white powder which was scraped from the sample of disintegrated concrete from the Shrewsbury River bridge pier, with the following results:

| | | |
|---|---|---|
| "Free Silica | $Si\ O_2$ | 57.64% |
| Alumina | $\left\{ \begin{matrix} Fe_2O_3 \\ Al_2O_3 \end{matrix} \right\}$ | 7.44 |
| Calcium | Ca | 6.83 |
| Magnesium | Mg | 8.66 |
| Chlorides | Cl | 1.82 |

"The large amount of silicia is undoubtedly due to the grains of sand which were scraped off the concrete with the sample. Eliminating the silica from consideration, we find that the sample is made up of approximately equal parts of magnesium, calcium, and iron alumina oxides. The magnesium is somewhat in excess of the calcium, which indicates a transfer of the calcium and magnesium bases between the cement in the concrete and the salt in the sea water. The magnesium of the sea water replaces the calcium of the cement. This action can only take place where the sea water has free access to the cement, or, in other words, at points where the concrete is quite porous. In a dense concrete such action could not take place, or would be so slow that it would be almost negligible."

At the south or up-stream end of one of these piers, when examined about 6 months ago, there was disintegration over a small area to a

Mr. Yates. depth of about 2 to 3 in. between high and low water. This end of the pier is very much exposed to damage from floating ice.

Another example of the deterioration of concrete in sea water is one of the piers of the bridge of The Central Railroad Company of New Jersey over the Elizabeth River at Elizabethport, N. J. Due to delays which were beyond control, it was necessary to deposit the concrete for these piers in freezing weather. Parts of the surfaces of the piers were injured slightly by freezing, which resulted later in a peeling of the surfaces. These surfaces were patched with a cement gun, and have shown no further peeling, with the exception that the patching on the down-stream end of one of the piers has fallen off. Where the injured surface of this pier is between high and low water the concrete has disintegrated to a depth of from 4 to 6 in.

The water of this stream is salt, but is apt to carry a large mixture of sewage and fresh water. The pier surfaces are covered with slime between high and low water, but, with the exception noted, show no deterioration such as might result from the action of salt water.

In regard to the construction of the New Hackensack and Passaic River Bridges by The Central Railroad Company of New Jersey in 1912-14, it may be of interest to note that, due to experience with the disintegration of concrete in sea water, it was decided to build those portions of the piers between high and low water with a granite facing, backed up by a concrete interior. This granite facing was carried from a point 2 ft. below mean low water to a point 2 ft. above high water.

Practically the whole of the 12 000 cu. yd. of concrete in the foundation of these piers was deposited in water with drop-bottom buckets. The channel piers extended to a depth of 30 ft. below low water, and the concrete was deposited continuously within timber coffer-dams up to a point 2 ft. below low water. About a day after the completion of the concrete, the water was pumped down, the laitance was removed from the upper surface, and the construction of the remaining portions of the piers was carried on in the air.

The concrete was composed of 1 part of Portland cement, 2 parts of washed Cow Bay sand, and 4 parts of washed and graded gravel.

In experimenting with different concrete aggregates, it was found that when Cow Bay sand and gravel were used, 2 to 3 in. of laitance would result in depositing a depth of 15 ft. of concrete under water. When Cow Bay sand and 1½-in. broken trap rock (dust screened out) were used, the result was about double that quantity of laitance. For this reason practically all the work was done with the gravel.

Examinations of the under-water surfaces by a diver, shortly after the completion of the structure, developed no defects, except that on one of the piers there was found a wedge-shaped horizontal band of laitance,

from 4 to 6 in. wide, a section of which the diver was able to remove to a depth of about 4 in. Investigations showed that this band of laitance was due to several days' stoppage of the work of depositing the concrete at this level, and that, although attempts had been made to remove the laitance with a clam-shell bucket and the help of a diver before the depositing was again started, the efforts were not entirely successful. Mr. Yates.

Annual general inspections of all the structures on the Central Railroad are made by the Bridge Engineer, and no disintegration similar to that found in salt water has ever been observed in concrete piers in fresh-water streams, although in many cases the concrete is exposed to damage from floating ice. Concrete surfaces have been found injured (probably by floating ice), but no disintegration has taken place. This rather disproves the theory that the disintegration of concrete in sea water, between tides, is due solely to the alternate freezing and thawing of the concrete.

The result of the speaker's experiences with the structures cited leads him to believe that concrete can be used safely in sea water, provided there is no danger of the surfaces being injured by floating material. Where such action is at all likely to take place, it has been decided to use a stone facing backed up with concrete. The speaker has no hesitation in using properly deposited concrete to a point 2 ft. below low tide, or to a height that is below any likelihood of injury by floating material.

Mr. McClintock.

J. R. McClintock,* M. Am. Soc. C. E.—The speaker had an experience, at Clarksburg, W. Va., with disintegration of some concrete beams across the top of an open coagulating basin. These beams are normally above the water level, but at times, owing to careless operation of the pumps, they are alternately submerged and exposed, causing in cold weather very severe freezing conditions.

The basin was constructed 7 years ago, and after about 5 years probably two-thirds of these beams were almost completely disintegrated, the concrete looking practically like a loose shale. The other beams, although under the same conditions, were practically intact.

As far as known, there was no difference in construction methods. They were all built by the same gang, with the same foreman, and presumably of the same materials. It is possible, however, that in some of these beams use was made of a local stone which is subject to disintegration when exposed to the weather. There were some delays in the shipment of the hard limestone regularly used, and it is very likely that some of the poor local stone may have been used, in order to avoid waiting.

The contractor on this work had rather a disappointing experience with the local stone. In excavating for a reservoir, probably 4 000

* New York City.

Mr. McClintock. or 5 000 cu. yd. of stone were piled up to be used later for concrete. After this material had remained piled up nearly a year, it was found to have weathered to almost a soft clay. When taken out it appeared to be a good hard stone.

The other concrete structures at Clarksburg, including a filter plant and a 1 000 000-gal. concrete tank, were substantially impervious to water, as judged by leakage measurements, and it is puzzling to understand just why the beams proved to be so porous as to allow the entrance of water to cause their disintegration.

There is another factor which should be mentioned. The river water receives a great deal of acid mine waste, and the sulphur compounds may have had some effect on the concrete. This, however, would not explain a selective action on some parts and not on others. Probably the best explanation is that poor stone was used in a portion of the beams, and good stone in the rest. The stone was of a nature between a shale and a sandstone. The different strata change gradually from a true shale into a sort of stone locally called sandstone.

Mr. Williamson. S. B. WILLIAMSON,* M. AM. SOC. C. E.—Before the construction of the locks on the Panama Canal, there were several independent investigations as to the effect of sea water on concrete, and all the information obtainable was gathered at that time. According to the speaker's memory, the failures recorded were due largely to porous or pervious concrete. On the strength of this information, the engineering authorities on the canal agreed to use concrete in the lock construction; and in the lower parts of the locks that were subject to sea water a richer mixture was used.

At Miraflores, where there is a 20-ft. tide, and the lock gates are about 83 ft. high, an interesting condition developed: It was found that the difference in the weight of the salt water on the lower side and the fresh water on the inside of the gates made it possible to have a head of about 18 in. on the lower side of the gates, tending to force them open when the lock was full.

In order to discover a means of overcoming this, a number of experiments were carried out, and it was finally ascertained that, by turning up the lower ends of the culverts so as to discharge at a certain elevation, the inter-mixture or diffusion took place in such a way as to equalize the pressure heads.

Thus far, the concrete in the locks has shown no signs of disintegration. There has been some disintegration, however, in the concrete docks built at Balboa. These are reinforced docks, supported by reinforced concrete cylinders, and some of the floor girders are partly submerged at extreme high tide. These show some disintegration, especially where cracks have appeared, but as yet there is nothing serious.

* New York City.

Great care was taken in the composition and placing of the concrete in the locks, particularly where they will be subject to sea water, and the speaker believes that the concrete is impervious and will not be readily attacked. Mr. Williamson.

WALDO C. BRIGGS,* M. AM. SOC. C. E.—There is one point which the speaker would very much like to have developed. If "the addition of clay to the cement had a slightly beneficial result" with the concrete subjected to such unusual conditions, why not modify the customary specification for sand, requiring it to be "clean and sharp", to read "clean and sharp, free from dirt, loam, mica, and organic matter, and shall contain not more than 5% by volume of clay"?† Mr. Briggs.

It happens sometimes that what appears to be a very good grade of sand is encountered in excavation made in connection with construction work; but if this sand contains clay, it necessarily is rejected if the specifications require it to be clean.

CHARLES S. BILYEU,‡ ASSOC. M. AM. SOC. C. E.—A curve of maximum density, or an ideal curve for the combination of sand and aggregate, has been plotted from the results of an extended series of tests by William B. Fuller and Sanford E. Thompson, Members, Am. Soc. C. E., at Jerome Park Reservoir, New York City, in 1902-04. Mr. Bilyeu.

The speaker's criticism of specifications refers particularly to those for reinforced concrete buildings. It is true that such specifications are frequently prepared in architects' offices, and that engineers are not responsible for them. They generally cover adequately the testing of the cement, but rarely contain any requirement for the analysis of the sand and aggregate.

The speaker has been called on, from time to time, to make tests of materials (particularly sand) used in large and important buildings. In several instances the sand has been found to contain an abnormally large percentage of very fine grains, and to be otherwise unsuitable for use in work of such character. Tests of this nature are usually arranged for only after the work is well under way and after it has been found that the concrete has not developed proper strength.

In the speaker's opinion, careful sand and aggregate analyses are very important investigations, and are absolutely necessary for safe and economical concrete construction.

W. E. DAY,‡ ASSOC. M AM. SOC. C. E.—In line with the discussion of sand mixtures, an incident on some construction work during the past season might be described. A sand pit was opened, about 2½ miles from the work, which contained a sand of excellent quality. The testing laboratory reported it to be very clean and that the percentages Mr. Day.

* Long Island City, N. Y.
†Bureau of Sewers, New York City.
‡New York City.

Mr. Day. of various sizes were such as to make it an almost perfect sand for concrete.

On account of the long haul, the experiment was made of mixing this pit sand with a very fine beach sand, containing considerable loam, which was right on the works. Test cubes and briquettes were made with pit sand containing from 0% to 45% of beach sand. These cubes and briquettes were tested after being stored 30 days, and the results showed that adding up to 30% of beach sand improved the strength of the concrete, as compared with concrete made from the so-called perfect sand.

Mr. Ridgway. ROBERT RIDGWAY,* M. AM. SOC. C. E.—It is generally agreed that the grading of sand is very important. A well-graded sand containing a small quantity of clay is preferable to a poorly-graded sand which is free from impurities, particularly if the sand is uniformly fine. It is well, however, to reduce the quantity of clay and other impurities to a minimum, and what one should strive to get is a clean, well-graded sand. The speaker does not agree with those who have advocated the use of a small quantity of clay to increase the density of the concrete. It is better to omit the clay and substitute a little more cement, in order to secure this density, thereby increasing the strength of the concrete as well. If the clay appears as lumps in the sand, it is obviously more objectionable, as Mr. Fuller states, than if in the form of a powder. The holes often seen in the surfaces of cement walks and in the faces of concrete walls are generally caused by the dissolving of lumps of clay which were incorporated in the mix.

Mention has been made of limestones as concrete aggregates. The use of limestones was not permitted in the Catskill Aqueduct specifications for pressure-tunnel lining because of their solubility. This was the result of a careful study of the subject by the Designing Division of the Board of Water Supply before the specifications were prepared. It was found, in a number of instances, that concrete had disintegrated, due to the solubility of the coarse aggregate. A notable example cited was that of the Thirlmere Aqueduct for the City of Manchester, England. In this case, as the speaker understands it, an almost pure limestone was used, and the soft water in the Aqueduct dissolved it to such an extent that the concrete was badly disintegrated. In most works, however, limestones are not prohibited; for example, the Public Service Commission specifications for subway construction in New York City permit the use of "Sound, hard, broken limestone".

In driving the new rapid transit tunnel under the East River from Old Slip, Manhattan, last year, the shield working under compressed air encountered the concrete foundation of the wall at the river bulk-

* New York City.

head line. This concrete had been deposited in bags under the salt water by the Department of Docks and Ferries of New York City about 10 years before. The mix was said to be 1: 2: 5, and it was found to be in excellent condition, as sound and dense as though it had been placed behind forms in the dry. The burlap of the bags was still there, but the concrete was bonded so well that it looked like good mass concrete. There was no sign of any disintegration. Mr. Ridgway.

GEORGE W. FULLER,* M. AM. SOC. C. E.—This discussion brings out four leading points: First, that imperviousness of concrete is a *sine qua non* for structures subjected to a fluctuating water line; secondly, laitance must be carefully guarded against; thirdly, there is the question of temperatures, in relation to disintegration by alternate freezing and thawing; and, fourthly, there is the question of what may be brought about by organic matters and their decomposition to form acids under putrefactive processes. Mr. Fuller.

Sanitary engineers know well, in connection with water-works and sewerage structures, what porous concrete may signify, especially as to the stability and integrity of structures above and near the flow line, as ordinarily seen.

Mr. McClintock has given some interesting observations on concrete sedimentation basins of the water-works plant at Clarksburg, W. Va. This shows what freezing and thawing may do, in the absence of any acids or organic matter to decompose the structures, where concrete, probably not as impervious as it should be, is subject to the various influences of the elements, especially where there is a fluctuating flow line.

Now for a few words in respect to the fourth element, and that is the disintegration of concrete, mortar, or brick, under those circumstances where the decomposition of organic matters of sewage or polluted water may bring about substances which produce acids.

That is probably an old story. It has been debated, year in and year out, in relation to the question as to whether concrete is as good as brick for large sewers.

It is a fact that the organic matters contained in sewage under some circumstances will undergo bacterial decomposition, and that the sulphur compounds in the organic matters will be changed to sulphureted hydrogen, in the complete absence of dissolved atmospheric oxygen, and in the presence of the right kinds of bacteria which ordinarily grow there. It is a further fact that, when bacterial agencies bring about what are commonly called putrefactive changes, sulphureted hydrogen and other products of putrefaction may be obtained from mineralized sulphur compounds, such as sulphate of lime or magnesia, and found in cement, fresh water, and sea water.

* New York City.

Mr. Fuller.

Sea water is so highly mineralized that we probably do not realize what opportunities there are for the formation of compounds of a dissolving nature. Neither do we know what inter-reactions may produce, in conjunction with putrefaction, as to solvents from magnesium chloride and other neutral salts of a supposedly stable and non-corrosive nature with respect to concrete.

The formation of sulphureted hydrogen does not cause an appreciable corrosion of fairly good concrete below the minimum water line of a sewer, septic tank, or pier of concrete, so far as the speaker's observations and studies go. Even in septic tanks, he has heard of only one instance of serious disintegration of concrete in the lower part of the structure, and that was in the case of a tank in the arid regions of the Southwest.

Sulphureted hydrogen must be converted into sulphuric acid in order to develop corrosive properties. This is effected when the gaseous sulphureted hydrogen rises above the water, meets the oxygen of the air, and in the presence of moisture. This is why sewers, septic tanks, and piers are disintegrated above and not below the flow line. Of course, this solvent action varies in intensity with the porosity of the concrete, effect of freezing, etc.

The outfall sewer of Los Angeles showed disintegration of mortar and signs of deterioration of brick in manholes beyond the main siphon, but not below the flow line. When the atmosphere with its oxygen was excluded, this disintegration was stopped to a surprisingly satisfactory extent.

Much sulphureted hydrogen does not escape into the atmosphere, as it combines with iron salts to form the black ferrous sulphide of iron, which gives liquids in process of putrefaction their black characteristic appearance. This is why the sludge of septic tanks is black, and why the sediment in pump wells and settling basins is black, as was reported in detail by the speaker some 20 years ago in his water purification investigations at Cincinnati.

Laitance should certainly be kept within reasonable limits, otherwise undesired results are to be expected. Undoubtedly, the pendulum is swinging back from the very wet mixes of recent years toward the dry mixes of 20 years ago. However, the mix should not be so dry as to make it problematical as to removing entrained air from the pores of the concrete.

It is feasible to use sand with a moderate quantity of silt or clay without vitiating the integrity of a structure, if that fine material can actually be used as filling uniformly a certain portion of all voids of the sand as a substitute for cement; but if such silt or clay is in lumps or is otherwise irregularly and improperly dispersed in the voids of the sand, the programme becomes invalid.

Fine material is objectionable if it occurs as a coating on the sand particles, as it weakens the concrete; or if it contains sufficient organic matter to interfere with the setting of the concrete; or if it is flaky or of such character as to produce lumps, on mixing the concrete. Mr. Fuller.

On the other hand, fine material may add materially to the imperviousness and density of the concrete. This is a function of the mix of concrete, the nature and quantity of the fine material; the percentage of voids in the sand; and the relative size and shape of the sand particles.

As Mr. Ridgway says, clean sand filled with an excess of cement has marked advantages in the direction of safety; but this is not always practicable in some localities, or worth its cost in others—as compared with using sand with fine material in it.

At York, Pa., the local building sand is a crushed rock containing 10% or more of fine material passing a 100-mesh sieve. It was used with satisfaction on several hundred thousand dollars worth of work on outfall sewers, pumping station, and sewage treatment plant. Apparently, this material did not form lumps. Its use saved the needless added cost of obtaining other sand transported from a long distance. However, this is a procedure which ought not to be adopted without careful investigation in each local case.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

### Discussion.*

BY MESSRS. E. LAUCHLI AND ROBERT A. SHAILER.

E LAUCHLI,† ESQ.—Those who have followed closely the progress made in driving Rogers Pass Tunnel cannot but admire the skill displayed by the engineers and contractors connected with this great enterprise; and, no doubt, the completion of this bore will give a greater impulse to work of this character in America. Mr. Lauchli.

The speaker, who was retained in an advisory capacity by one of the contractors bidding on this work, was well aware of the conditions preceding the driving of this tunnel. In his opinion, bidding on this work was a gambling proposition without precedent, owing to the fact that the railway company did not provide the bidders with sufficient information relating to the geological formation of the range to be tunneled. The accuracy of this statement is backed by the fact that, according to Mr. J. G. Sullivan, Chief Engineer of the Canadian Pacific Railway, expert tunnel contractors bid on this work from $8 to $11.25 per cu. yd. for excavation—prices hitherto unheard of in connection with double-track tunnel excavation.

That the driving of this tunnel has been a success will not be denied by the Profession, but, in the speaker's opinion, the results attained are due more to the unusually favorable conditions—as far as material penetrated, absence of rock pressure, small quantity of water underground, and low rock temperatures are concerned—than to the driving methods adopted by the contractors.

* This discussion (of the paper by A. C. Dennis, M. Am. Soc. C. E., published in January, 1917, *Proceedings*, and presented at the meeting of February 7th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr Lauchli. In preparing their bids, those interested in this work, had to assume that there would be a reasonable amount of difficulty, and this, no doubt, accounts for their selection of the bottom-heading method of tunneling, the only method known to-day which insures absolute success, both under favorable and unfavorable conditions. Now, as just mentioned, there was no information available of a nature to cause bidders to select a driving method, practicable under ideal conditions only, and no experienced tunnel contractor could afford to gamble on an undertaking of this magnitude.

The driving method actually adopted based its greatest expectations on the enlargement of the tunnel, and necessitated an auxiliary or pioneer tunnel. Ventilation alone did not call for an exceptional ventilation system, as has been proved during past years, in connection with the driving of tunnels from 5 to 9 miles long, driven under far less favorable conditions than those at Rogers Pass.

Had an intense ground pressure developed over a long section, or over several short sections, then in order to keep within the schedule of progress, the method of enlarging the bore at one point only would have necessitated such radical changes as to jeopardize the success of the enterprise. In Rogers Pass Tunnel it was possible to enlarge the tunnel at one point only, and an obstacle, such as intense ground pressure at the enlargement point, would have necessitated keeping the timbering, and probably also the masonry lining, close to the enlargement; and thus the blasting and mucking operations would have been greatly hampered.

On the other hand, the method of enlarging the bore to full section from several working points, as used for instance in the Loetschberg, Hauenstein, and Grangs Tunnels in Switzerland, possesses the advantage of being adaptable both in light and heavy ground, and, under favorable conditions, such as met in Rogers Pass Tunnel, the drilling and blasting operations are at least as economical as those used in the last bore referred to, the only differences being the temporary heading timbering and the removal of the two side benches adjacent to the heading.

It may be useless to point out here that the removal of the benches could have been done with air-operated shovels. It is questionable, also, whether the total cost of driving this tunnel by the method actually adopted was less than it would have been by using the bottom-heading method. In the first place, the pioneer or auxiliary tunnel method involves practically two headings instead of only one, as used in connection with the European method; for, in this case, the length of the pioneer tunnel, together with the cross-cuts, is a little more than 20 000 ft., and, of course, this auxiliary tunnel is chargeable to the main bore. Then, also, it is questionable whether the weathering or ground pressure will not incur maintenance charges

in connection with the pioneer tunnel and the cross-cuts, in order to avoid disturbances of the rock surrounding the main bore. The practice, also, of not lining the main bore at the very outset is open to criticism, as it is always desirable to prevent initial disturbances from taking place in the overlying strata, especially in deeply overlaid tunnels, rather than to check these later, perhaps under operating conditions in the tunnel. Mr. Lauchli.

The speaker, who made a careful study of the proposition, figured that, on the assumption of a daily progress of 20 ft. from each portal, and assuming that one-third of the tunnel would necessitate temporary timbering, also that the masonry lining would follow close to the excavation, the cost of driving this bore, with a bottom heading, would be \$5 per cu. yd., exclusive of contractor's profit. Mr. J. G. Sullivan has stated* that the cost of driving the tunnel through rock, including the cost of driving the pioneer tunnel and the cross-cuts, "will amount to a little less than \$5 per cu. yd. for excavation in the tunnel proper."

In comparing the costs of different methods, the mistake has often been made of comparing short tunnels with long ones, such as the Simplon or the Loetschberg; for, it is a well-known fact, that the cost per linear foot of long tunnels is higher than that of short ones. For instance: for the first kilometer of the Loetschberg Tunnel, the cost was \$160 per lin. ft. including lining; the cost for the seventh kilometer (center of tunnel) was \$224 per lin. ft., the average for the whole bore being \$201.65 per lin. ft.

On the other hand, the cost of Rogers Pass Tunnel can be compared favorably with that of the more recently completed Hauenstein Tunnel, 26 680 ft. long, driven under conditions very similar to those encountered in Rogers Pass Tunnel, although less favorable. The wages paid in connection with the Hauenstein Tunnel were higher than in the case of the Loetschberg Tunnel, being about half as much as those paid at Rogers Pass Tunnel.

On the other hand, the tunnel was driven 20 000 ft. from the south portal, the remaining 6 680 ft. being driven from the north portal, and this alone increased the cost. Another feature of material importance is the fact that the bore was lined throughout, the masonry lining being kept close to the excavation. This feature also complicates driving operations, and increases the cost of excavation by about 50 cents per cu. yd.

Finally, the mining operations had to be carried on very carefully on account of swelling ground, and at several points masonry inverts had to be provided; yet the cost of excavation was less than \$2.70 per cu. yd. Had it been possible to line this tunnel after the completion of the excavation, and, also, had the tunnel been taken out equally

* *Engineering News*, February 24th, 1916.

Mr. Lauchli. from each end, there is no doubt that the cost for excavation would not have been more than $2.25 per cu. yd.

This bore was driven at an average speed of 750 ft. per month for the heading, for a whole year (maximum 1 030 ft.), and at the rate of 658 ft. per month for stoping (maximum 870 ft.), also at a rate of 656 ft. per month for enlarging to full section (maximum 850 ft.); and the masonry lining was carried on at the rate of 650 ft. per month.

Mr. Shailer. ROBERT A. SHAILER,* M. AM. SOC. C. E.—The speaker is of the opinion that the pioneer heading, entirely outside the main tunnel section, is a new feature in tunnel construction.

Apparently, the time of completion was vital, and the fact that the contract was completed more than 11 months ahead of the specified time surely shows that the adoption of the pioneer heading was justified economically.

To the advantages which Mr. Dennis claims for this heading there might be added: that it enabled the work on the long rock tunnel to be well under way before the earth and soft ground approaches on the east and west ends were commenced; also, that the pioneer heading served the purpose of a pilot tunnel to inform the contractor as to the character of the rock through which the main tunnel was to be driven.

Had the speaker's opinion been asked as to the plant layout before the work was commenced, he would have said that five boilers, each of 150 h. p., would prove inadequate to fulfill the requirements at each end of the tunnel, namely: 4 000 ft. of free air compressed to 100 to 125 lb., and 1 500 ft. of free air compressed to 1 000 to 1 100 lb., as well as power for the lighting, ventilating, and pumping plants; and apparently, that opinion would have been—in some degree, at least—correct, for, the author says: "The boilers were run generally much beyond their nominal rating", and this, as all know, is not good for the boilers or for the contractor's pocketbook.

It is noted that there was a drop of approximately 15% from the power-house pressure to the drills, and of 25% to the high-pressure charging station, due probably to the pipe lines being of too small diameter. This loss seems excessive, and must have proved expensive.

The descriptions of the various tunnel operations are clear and concise, and the speaker thinks the members of the Society are indebted to Mr. Dennis for this paper.

* New York City.

## AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852


# PAPERS AND DISCUSSIONS



## DISCUSSION ON REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES*


By J. E. Willoughby, M. Am. Soc. C. E.


J. E. Willoughby,† M. Am. Soc. C. E. (by letter).‡—In its conclusions on the Fundamental Principles of Valuation,§ the Committee fails to direct especial attention to the fact that the principles and methods must be such as will insure to a property subject to competition an opportunity to benefit in what is sometimes called the unearned increment, in order to attract capital to the service of the public. The investor in a railroad property must have a hope of gain beyond the current rate of interest, because there can never be a guaranty that a railway (as it is always subject to competition) will earn the current rate of interest on its "fair value". Mr. Willoughby.

The Committee's classification of "New Properties", "Old Properties under Continuous Regulation", and "Old Properties not Under Continuous Regulation", leads to the assumption that the Committee considers the fair value of the property to be related in some way to past accounting. Such a conclusion is unfortunate. The fair value of a property is not affected in any way by the accuracy or inaccuracy of the accounting. The book record only, of the cost of a property, is thus affected. Even if the capital expenditure accounting were correctly kept throughout the history of a property, the accounting would

* Presented to the Annual Meeting, January 17th, 1917.

† Wilmington, N. C.

‡ Received by the Secretary, January 29th, 1917.

§ Page 1716, *Proceedings*, Am. Soc. C. E., for December, 1916.

Mr. Willoughby. show merely the monies expended on it, less retirements, and the accrued depreciation, if any, and the accumulated assets. The accounting will not show the fair value of the property.

The writer is not in sympathy with the idea that a property has one value for rate-making purposes, another value for capitalization, and yet another for taxation. It is quite true that property of all kinds is returned for taxation at less than its market value. Some States—*e. g.,* Alabama—fix by statute what percentage of the market value shall be subject to tax levy; in other States, such a percentage is fixed by common consent. It is also true that it will be imprudent for the operating management of a steam railway property to capitalize many items of expenditure which the Interstate Commerce Commission Accounting Classification permits—*e. g.,* the assessment against the railway for a sewer—because the expenditure, being in the nature of a tax, ought to be provided for out of income. Only those expenditures should be capitalized by the operating management as may increase the revenues of the railway; as may lessen the operating expenses; or as may be conducive in some measure to the safety, comfort, or convenience of the employees or patrons of the railway. The rates fixed by and for the railway should be sufficient to provide for expenditures out of income to cover all expenditures which do not come within this classification.

The exception which the Committee makes in its list of "Physical Property to be Included in Valuation" (page 1717) under *"Excessive Size or Capacity"* is not consistent. The Committee correctly proposes to value the actual property as existing on the day of valuation, and not a substitute property serving the same purpose. The Committee should not vest the engineer in charge of the valuation with the power of reviewing the errors of judgment of the original construction, because that is but a step toward valuing the theoretical property, which will serve the same purpose, and is abhorrent to the Committee's views. Under the exception, much of the present railway construction will be eliminated in part from a valuation. There are noteworthy examples in the Union Passenger Station at Washington, the Pennsylvania Terminal in New York City; and in abnormal expenditures for low grades and multiple tracks in some recent railway construction.

The writer will add, as a corollary to the paragraph "Title to Property not Conclusive", that all structures on land to which the owner of the public utility has title should be included in the valuation. The inventory of land will necessarily include all lands to which the owner has paper title, and not held adversely by others; and also all lands which the owner holds adversely to others, the owner being without paper title. The theory of the law, that ownership of land covers ownership of all fixed improvements, gives to the railway owning the land all fixed improvements thereon, although those improvements may

have been paid for wholly or in part by others. This view is not inconsistent with the theory of cost of reproduction which requires us to consider the railway under investigation as the junior line. Deeds to lands and the possession of lands in absence of deeds are the physical items of an inventory of lands, just as the cross-ties, rails, and fastenings are physical items of an inventory of track. In an estimate of cost of reproduction, all structures and appliances that exist to enable the railway under investigation (called *A*) to cross another railway (called *B*) must necessarily be inventoried in favor of the railway *A*, regardless of ownership of land, because such expenditures are a part of the cost of crossing. The railway *A*, under investigation, being non-existent, must provide such structures and appliances as will enable railway *A* to surmount the physical obstacle which exists as railway *B*. In like manner, when railway *B* is under investigation, the cost of structures and appliances to cross railway *A* must be inventoried in favor of railway *B*. This double inventory is not contrary to the public interest, because the public benefits enormously in the reduction of unit costs from the assumption of the existence of all railways except the railway under investigation, and suffers only a very small addition to the collective estimate of cost of reproduction of all the railways. The writer has noticed with impatience the recent discussion of methods for dividing the cost of crossing structures between the two railways on some historical basis, because such discussion contemplates merely a way to prevent the assessment twice of a very small item of the cost of reproduction, and at the same time to lessen greatly the actual historical costs of the property under investigation in all items that the presence of the railway to be crossed will affect. Mr. Willoughby.

The Committee is correct in its conclusion on "Cost of Reproduction" (page 1719) that the identical property is to be reproduced, and not a substitute, and that normal present conditions shall determine the prices and methods for doing the work. It is worth while to review just what these conclusions require for railway valuation:

1.—The rail and fastenings, ties, ballast, and track accessories must be considered as new, and of the quality inventoried. It is true that many branch lines of the greater systems were laid with relay rail on the date of first construction, but such practice was a result of the development of the system. It will be inconvenient to go into this feature of development expense—*i. e.*, to assess the losses to the railway due to the removal of rail from principal lines, before the full life of the rail had been obtained, so that relay rail for branches might be available. Considerable quantities of relay rail are not sold in the market for new construction, and therefore it is inconsistent with the theory of reproduction to estimate

Mr. Willoughby.

relay rails for branch lines when the material is not available for purchase for such works. By assessing the decretion to the rail fastenings and track accessories in branch lines, from new to the date of inventory condition, the interest of the public will be fully protected. Many sections of the country are without suitable ballast, and ballast is necessarily handled for long distances, over tracks that have been surfaced on a temporary ballast. A haul is now being made, on gravel ballast to points in Florida, that averages 490 miles. The unit costs must cover the haul. The construction programme will provide properly for the use of temporary ballast, and the hauling and placing of the permanent ballast after completion of the track laying, but the time limit for construction must be greatly extended, and special equipment must be provided for. It will be improper to consider that the entire energies of the railway will be exerted in hauling ballast. The ballast movement will be subject to the delays of the railway's date of valuation traffic—that traffic being one of the present-day conditions.

2.—The cuts and fills must be of the dimensions and of the materials shown on the date of valuation. A fill made of rock or of clay in a sand country must be so inventoried, regardless of whether the adjoining country exhibits only sand. On many of the railways in the Atlantic Coastal Plain, where the soils are largely sand, clay has been hauled many miles, and at great cost, to aid in making a stable embankment. A cut that, at the date of the inventory, is too narrow for steam shovel operation must have unit prices affixed commensurate with the cost of removal by wagons, carts, scrapers, or hand, and must have a time limit sufficient for these slower methods. The cuts in Florida sand (which is too light to support teams) must be considered as being removed by hand (or by steam shovel if the dimensions warrant), and must have unit prices affixed on such basis. Where present-day construction methods still provide for the two-way basis of pay for earthwork, the two-way basis must be assumed. Embankments through the swamps of the Southeast, which will require trestles for construction, on account of lack of available borrow, must have that feature of cost considered. Especial attention must be given to inventory the condition of the material in the top surface of the roadway. All the older railways, by the tamping action of traffic, and by the provision of select soils for track surfacing in past years, have built up the roadbed into a dense crust, impervious to water, and of great sustaining power for the ballast. This crust is an asset to be inven-

toried. It is not usually obtained in present-day construction because ballast is put directly on the soft roadbed, with resulting water pockets and other objectionable features. A rock cut in a town or city must have unit prices large enough to cover the extra cost of taking the rock out so as not to injure persons and property adjoining, although the town or city may have been built up about the cut after the latter was actually excavated. Borrow from an embankment must be taken from points now available. The inventory must locate available borrow-pits. Borrow cannot be assumed as being taken from improved city property, although the borrow may have been originally obtained from the grading of vacant lots before the city was built. Mr. Willoughby.

3.—The bridges, trestles, and culverts must be of the dimensions and material shown on the date of inventory. Where stone, sand, and gravel for masonry is available in quantities necessary for the completion of the work within the adopted time limit only at great distances from the work, these materials must have unit prices sufficient to cover the cost of hauling from those distant points. Concrete is now being built on a new line in Central Florida, for which the gravel is hauled 600 miles, and for which sand is hauled 200 miles, because there is neither suitable gravel nor sand for concrete available in the territory through which the new line is being built.

4.—The right of way of the railway to be valued is cleared and usually grubbed. Proper consideration of these items requires that they shall be assessed on the basis of the condition of adjoining lands. If the adjoining lands be fields that are now cleared and grubbed, the assumption lies that the right of way will be cleared and grubbed, and *vice versa.*

5.—Due consideration must be given to the normal resources of the country as to labor and supervision. Labor is limited in amount, and any great increase in demand for labor causes the wages to rise and carries increased unit costs. The construction programme must not be shorter than the normal labor resources warrant.

6.—The annoyances, expenses, and delays of and to the conduct of construction works under present-day public antagonisms, political chicane, and legal proceedings must all be incorporated in the costs of reproduction.

The writer is not in accord with the Committee that "history must also be considered, to determine what is to be reproduced, the conditions under which it is to be reproduced, and how estimates must be made". History has no place in an estimate of the cost of reproduction

Mr. Willoughby. of a railway, except in so far as history is reflected in the experience of the engineer who makes the valuation. In a railway, the items to be reproduced are capable of being inventoried to the extent that such items are now duly inventoried in the construction engineer's estimate in advance of actual construction of present-day works. The idea underlying cost of reproduction is to fix: For what money can the existing property be built now? To mix the past with the present will only confuse the issue.

In making a proper financial and construction programme, unlimited capital must not be assumed. All our large systems of railways have been built up slowly, and the capital required has been available generally on reasonable terms. The tendency of engineers in fixing a programme for reproduction is to consider the physical items only as affecting the time limit. Experience does not justify ignoring the effect which capital requirements will have on the time limit, the unit prices, and the total cost of the property.

There is a value due to continuity which attaches to a railway right of way, but this value is not to be considered in connection with the cost of reproduction. It must be considered as a part of the fair value of the property when that value is fixed.

The writer expresses his approval of the distinction made by the Committee between "decretion" and "depreciation" (page 1722). For a railway, the Committee might have brought out more clearly the fact that decretion is an operating expense which for the older railways is taken care of by annual renewals, and which is invariably more than offset by assets of the railway other than those entering into a cost of reproduction valuation.

The Committee's conclusion on depreciation (page 1726) is a correct one. The words "funds held in reserve for such property", mean, of course, securities as well as cash.

It is well to remember that, in ascertaining the value of accrued depreciation, the basis is not the original cost of the item, but the replacement cost. As an example, the labor cost of track laying and surfacing on new lines varies from \$1 000 to \$2 000 per mile. The labor cost of relaying the rail and fastenings, chargeable also to track laying and surfacing under the Interstate Commerce Commission Classification, varies from \$200 to \$500 per mile. It is proper to apply decretion to track laying and surfacing, but only to the sum necessary to relay. In some of the tentative valuations of railways lately issued by the Interstate Commerce Commission, depreciation is applied to the cost of track laying and surfacing as of new lines, and the accrued depreciation so determined is in excess of the total cost of relaying, which means that the decretion is more than 100%, and is absurd.

Appreciation is an asset to be added to cost of reproduction new. The Committee is correct in the conclusion that appreciation is not to

be used as an offset to depreciation. This conclusion is true because appreciation, being an item to be inventoried and valued as a part of the cost of reproduction new, cannot be used as an offset, any more than other items of the physical make-ups of the property can be used as such an offset. The solidification, seasoning, and adaptation of a railway is as real an item of construction cost as is the cost of haul of material from cut to fill. Mr. Willoughby.

"Development Expenses" (page 1727) should properly include the losses due to keeping the property up to the requirements of public use. Many items of permanent railway construction are abandoned because substitute structures must be built to meet the public needs. As an example, consider the abandonment, by the railways of the Southeast, of pile foundations for wooden trestles when public service forced the substitution of 10-ft. trestle spans for 12 and 13-ft. spans. There are single systems in the Southeast with more than 70 miles of wooden trestles. The foundation piles abandoned as a result of the public requirements will cost not less than $10 000 per mile. The abandoned property, being wood permanently moist, has a life expectancy that is practically perpetual.

be used as an offset to deterioration. This expenditure [illegible] improvement being an increase in [illegible] the cost of [illegible] than [illegible] as the [illegible] can be used as soon as [illegible]. The [illegible] is a great [illegible] of land of national [illegible].

The [illegible] should [illegible] the [illegible] the property or to the maintenance of public use. [illegible] can be used to [illegible]. As an example, consider the [illegible] of [illegible] 12 and 13 ft. [illegible]. There are single [illegible] with more than 75 miles of wooden trestles. The foundation piles abandoned as a result of the public requirements will cost not less than $10,000 per mile. The abandoned property [illegible] permanently [illegible] permitted.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852


# PAPERS AND DISCUSSIONS



## DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE TO INVESTIGATE THE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS*

By Messrs. H. S. Schick, Lewis A. Jones, and R. S. Wise.

**H. S. Schick,† Assoc. M. Am. Soc. C. E. (by letter).‡—This Committee has performed a valuable service in a matter which is of first importance in the work of the Society. The diagrams accompanying the report are interesting, and their scope and limitations have been fully discussed by the Committee.** **Mr. Schick.**

**An important principle brought out in the report should receive further attention, for the purpose of emphasizing the helpfulness of the Society in promoting effectually the feeling of reciprocal usefulness, the spirit of co-operation between members, and, at the same time, perform a service of especially practical value to the profession.**

**The writer refers to the principle that the compensation of every engineer, aside from his return on physical capital invested, depends entirely on his own efficiency—both his technical and his selling efficiency. All know, on the one hand, as stated clearly by the Committee, that engineers frequently receive a lower compensation than their services should demand, and, on the other hand, that the search for the proper man to perform a given service is always attended with some difficulty and much loss of time. This difficulty may be charged to the fact that, in marketing their services, engineers are not always good salesmen, and have not the most ready means to acquire reliable**

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* Presented to the Annual Meeting, January 17th, 1917.

† Wheeling, W. Va.

‡ Received by the Secretary, January 15th, 1917.

Mr. Schick. disinterested information regarding the demand for their work. For a number of years, the American Society of Mechanical Engineers has maintained a bureau for the purpose of gathering and publishing, in several ways, just such data for the use of its members, at a cost which is surprisingly low, and with highly gratifying results.

This matter is brought to the attention of the members of this Society, now, for the purpose of eliciting discussion, and perhaps ultimately turning the valuable contribution of this Committee toward the accomplishment of a practical end which will be of much satisfaction to all.

Mr. Jones. LEWIS A. JONES,* ASSOC. M. AM. SOC. C. E. (by letter).†—There is one point in this report to which the writer desires to call attention. In Diagram 4,‡ giving the average yearly compensation grouped with reference to nature of service, the curve for employees of the National Government is shown to increase in a fairly regular manner, with an average yearly compensation of $2 899. Attention is called to the fact that no distinction is made between army and navy engineer officers and civilian employees. It is a well-known fact that engineer officers in the Government service receive practically twice as much as civilian employees, counting the allowances for quarters, etc., and, as a result, the curve, in its present shape, does the civilian employees an injustice. Could not a separate curve be made for each? The writer believes that the results would show the engineer officers' curve following closely the curve labeled "Private Companies", and that for the civilian engineers well below the one labeled "States and Counties".

Mr. Wise. R. S. WISE,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—This report is very complete and interesting, so far as it relates to the civil engineer, and the Committee deserves a great deal of credit for its work.

The writer is sure that a great many of our members would like to see a committee appointed to investigate and find ways and means of regulating the fees which the municipal engineer and surveyor should get for his services. As it is now, there is no schedule of fees on which such a man may base his charges.

On account of keen competition, he must base his charges on local rates, which in most cases have been established for many years; but, if rates were established, clients would not go shopping to see which surveyor would do his work the cheapest.

There should be fixed rates for establishing grades, cross-sections, and calculations of quantities on street work, for giving lines and grades for curbing, guttering, flagging, macadamizing, etc. Minimum

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* Montgomery, Ala.

† Received by the Secretary January 6th, 1917.

‡ *Proceedings*, Am. Soc. C. E., for December, 1916.

§ Passaic, N. J.

‖ Received by the Secretary January 13th, 1917.

rates for staking out lots, based on the size of the lot, for surveying and mapping out land, based on the location; one rate for the fire or congested district and another for the rural section. Mr. Wise.

There should be some understanding as to Court charges, not strictly for professional work, but for the surveyor's knowledge as to certain lines, maps, etc. In the district around Passaic, N. J., it has been the habit of some lawyers to employ the surveyor to locate certain properties, and later to subpœna him, often keeping him in Court two or three days, breaking up his field party, and not paying him for his time in Court.

rates for making out deeds, based on the size of the lot, for surveying and mapping out land, based on the acreage; one rate for the fire or congested district and another for the rural section.

There should be some understanding as to what charges, not dictated by professional worth, but for the surveyor's knowledge in a certain locale, etc. In the district around Toronto, it has been the habit of some lawyers to consult the surveyor as to [illegible] [illegible], and have to [illegible] from [illegible] in Toronto, two or three days, [illegible] to his field party, and not [illegible] him [illegible] in cheques.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By A. H. Rhett, Assoc. M. Am. Soc. C. E.

A. H. Rhett,† Assoc. M. Am. Soc. C. E. (by letter).‡—*Integral Water-Proofing of Concrete.*—In this valuable and interesting report§ the elusory subject of the water-proofing of concrete, or rendering it impervious to the percolation of water, again appears, and again invites discussion. Mr. Rhett.

Reduced to its most practical form, the question becomes merely whether or not concrete can be made non-porous; and, if so, what is the simplest, most effective, and cheapest method of doing it.

The first step in this direction should be, and is, the proper proportioning of the ingredients to fit so closely and compactly that no voids remain, and much experiment and investigation has been done along these lines. Repeated experiment, however, has seemed to show that, with the best possible grading, tamping, and mixing with available aggregates, there still remains from 10 to 15% of voids, while there is also an added percentage due to pocketing of water or air during the mixing and placing process.

It is not essential, of course, to close every void to render the concrete water-proof, as far as percolation through it is concerned, because continuity of the voids will play a considerable part in any percolation; but the rendering of the concrete absolutely non-porous is desirable sometimes for other reasons. The entrance into the pores

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* Presented to the Annual Meeting, January 17th, 1917.

† New York City.

‡ Received by the Secretary, January 11th, 1917.

§ *Proceedings*, Am. Soc. C. E., for December, 1916.

Mr. Rhett. of water affected with acid or alkaline properties gives rise to disintegration of the concrete, from the reaction of the acid or alkali and the free lime always present, and the preservation of the integrity of the concrete is more important than the actual water-proofing of it. Furthermore, any such reaction would eventually render the concrete pervious, even if it were not so originally.

The problem then, in its final form, becomes one of closing this 10 to 15% of voids that remain after the best proportioning has been obtained and those that occur from the pocketing of water or air in the concrete during the mixing process, or from friction between the particles of the aggregates in placing; and a corrective must be sought in means other than proportioning.

A study of the hydration of Portland cement seems to show that such a correction lies latent in the cement itself, and can be made available under proper treatment.

It has been shown* that the so-called beta-orthosilicate element of Portland cement is very slow to hydrate, and that but little trace of its hydration is found in any cement before 7 days, and in some cases it does not appear until 28 days, but that when this hydration does commence it progresses as long as water is present, and with a great increase in volume. This increase in the volume of Portland cement when continually mixed with water has been also strikingly shown by Nathan C. Johnson,† Assoc. M. Am. Soc. C. E.

This action of the cement, of course, must occur in concrete where the mixing is thorough and when water is held in the mix long enough to permit it. A further point to note is that chemical action of this sort is retarded by pressure, and would be most active in the direction of least resistance and consequently into the void spaces. It is also an interesting fact that this beta-orthosilicate element hydrates with an amorphous and not a crystalline structure. If the latter were the case, there would be a tendency to disrupt the concrete, but the amorphous texture of the increased volume enables it to adjust itself to the void forms and fill them.

It is a well-known fact in daily practice, and has recently been again noted,‡ that, with a given mix, the strength, and therefore density, will vary directly with the length of time required to dry out.

It would seem entirely possible, therefore, theoretically to get a non-porous concrete:

(1)—If the aggregates were properly proportioned;

(2)—If the mixing were thorough;

(3)—If the placing and tamping were properly done;

* *Technologic Paper No. 43,* U. S. Bureau of Standards.

† "Materials *vs.* Methods—Testimony of Moving Pictures in the Study of Concrete," *Engineering Record,* December 4th, 1915.

‡ "The Strength and Other Properties of Concrete as Affected by Materials and Methods of Preparation," *Technologic Paper No. 58,* U. S. Bureau of Standards.

(4)—If the quantity of water used were correct; and Mr. Rhett.

(5)—If the forms were kept on long enough to insure the retention of sufficient water to hydrate the cement thoroughly.

Any practictal man knows the difficulty and expense, if not the impossibility, of obtaining these conditions, and it becomes a very practical question as to whether or not they might not be obtained by a less expensive and more automatic or fool-proof method.

Answer to this question has been sought in the so-called integral water-proofing compounds, and they have been produced on several theories.

The simplest was the addition of some very finely particled inert material that would fill the voids. There seems to be no reason to doubt that the addition of such material might, to a certain degree, compensate for a faulty proportion of aggregates, but could scarcely be relied on to fill such voids as are caused by the mechanical pocketing of air and water in the concrete during mixing, or by the friction of the particles in placing.

Another theory has been to mix with the concrete a water-repellent material that will coat the voids and pores and prevent absorption. It is difficult to see, though, how such a material could coat the pores without coating also the particles of cement and preventing proper hydration, or how it could be relied on to coat every pore.

A better and more rational theory, however, has been to add a material which will lubricate the aggregates, and thus enable them to settle in a denser mass by eliminating the friction, and which will also act as an inert void filler. Such a material would eliminate the necessity for much tamping, and would compensate to a certain degree for a faulty proportion of aggregates; but there would probably still remain a certain percentage of voids due to the air and water pockets formed in the mixing. These, it would seem, can really only be eliminated by the swelling action caused by the full hydration of the cement, as described previously.

If, however, a material could be added to the concrete that would not only act as a lubricator and void filler, but, in addition, would tend to hold the water in the concrete and prevent too quick drying, it would certainly seem—on perfectly rational grounds—to compensate to a considerable degree for faulty proportioning, careless placing and tamping, inadequate mixing, and early removal of forms.

Such action, moreover, would be automatic, and might well prove much less expensive and more practical than the conditions necessary to obtain the same result without the addition of the compound.

The object of this discussion has been to show, if possible, that there is a theory which is rational and will explain the success that has attended the use of some of these compounds, in so many instances,

Mr. Rhett. and that the practical evidence of their effectiveness cannot be candidly disregarded.

To obtain this success, these materials should be used intelligently. There is nothing miraculous about their action. They cannot compensate for cracking or breaking of the concrete, for very lean mixes, or for an acid or alkaline content in the sand, but they can and do compensate, to a degree which is eminently practical, for faulty proportioning, careless placing and tamping, improper mixing, and too quick removal of the forms.

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# PAPERS AND DISCUSSIONS



## APPENDICES
## TO THE
## REPORT OF THE SPECIAL COMMITTEE
## TO FORMULATE PRINCIPLES AND METHODS
## FOR THE VALUATION OF RAILROAD PROPERTY
## AND OTHER PUBLIC UTILITIES

# APPENDIX I.

## ACCOMPANYING REPORT OF THE SPECIAL COMMITTEE ON VALUATION OF PUBLIC UTILITIES.

## DEPRECIATION TABLES,

BASED ON COMPOUND-INTEREST METHOD,
HERETOFORE CALLED EQUAL-ANNUAL-PAYMENT METHOD.

INTEREST COMPOUNDED ANNUALLY.

(" Value " is value at end of year ; " Dep." is depreciation during year.)

### 5-YEAR LIFE.

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 0 | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 | |
| | | 18.4627 | | 18.0975 | | 17.7396 | | 17.3891 |
| 1 | 81.5373 | | 81.9025 | | 82.2604 | | 82.6109 | |
| | | 19.2012 | | 19.0023 | | 18.8041 | | 18.6063 |
| 2 | 62.3361 | | 62.9002 | | 63.4563 | | 64.0046 | |
| | | 19.9693 | | 19.9525 | | 19.9322 | | 19.9087 |
| 3 | 42.3668 | | 42.9477 | | 43.5241 | | 44.0959 | |
| | | 20.7680 | | 20.9501 | | 21.1282 | | 21.3024 |
| 4 | 21.5988 | | 21.9976 | | 22.3959 | | 22.7935 | |
| | | 21.5988 | | 21.9976 | | 22.3959 | | 22.7935 |
| 5 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

### 10-YEAR LIFE.

| Age, in years. | Value. 4% | Dep. 4% | Value. 5% | Dep. 5% | Value. 6% | Dep. 6% | Value. 7% | Dep. 7% |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 | |
| | | 8.3291 | | 7.9505 | | 7.5868 | | 7.2377 |
| 1 | 91.6709 | | 92.0495 | | 92.4132 | | 92.7623 | |
| | | 8.6623 | | 8.3480 | | 8.0420 | | 7.7444 |
| 2 | 83.0086 | | 83.7015 | | 84.3712 | | 85.0179 | |
| | | 9.0088 | | 8.7654 | | 8.5245 | | 8.2865 |
| 3 | 73.9998 | | 74.9361 | | 75.8467 | | 76.7314 | |
| | | 9.3690 | | 9.2037 | | 9.0360 | | 8.8666 |
| 4 | 64.6308 | | 65.7324 | | 66.8107 | | 67.8648 | |
| | | 9.7439 | | 9.6638 | | 9.5782 | | 9.4872 |
| 5 | 54.8869 | | 56.0686 | | 57.2325 | | 58.3776 | |
| | | 10.1336 | | 10.1470 | | 10.1528 | | 10.1513 |
| 6 | 44.7533 | | 45.9216 | | 47.0797 | | 48.2263 | |
| | | 10.5389 | | 10.6544 | | 10.7620 | | 10.8619 |
| 7 | 34.2144 | | 35.2672 | | 36.3177 | | 37.3644 | |
| | | 10.9606 | | 11.1871 | | 11.4078 | | 11.6223 |
| 8 | 23.2538 | | 24.0801 | | 24.9099 | | 25.7421 | |
| | | 11.3989 | | 11.7464 | | 12.0922 | | 12.4358 |
| 9 | 11.8549 | | 12.3337 | | 12.8177 | | 13.3063 | |
| | | 11.8549 | | 12.3337 | | 12.8177 | | 13.3063 |
| 10 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

### 15-YEAR LIFE.

| Age, in years. | Value. 4% | Dep. 4% | Value. 5% | Dep. 5% | Value. 6% | Dep. 6% | Value. 7% | Dep. 7% |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 | |
| | | 4.9941 | | 4.6342 | | 4.2963 | | 3.9795 |
| 1 | 95.0059 | | 95.3658 | | 95.7037 | | 96.0205 | |
| | | 5.1939 | | 4.8660 | | 4.5540 | | 4.2580 |
| 2 | 89.8120 | | 90.4998 | | 91.1497 | | 91.7625 | |
| | | 5.4016 | | 5.1092 | | 4.8273 | | 4.5561 |
| 3 | 84.4104 | | 85.3906 | | 86.3224 | | 87.2064 | |
| | | 5.6177 | | 5.3646 | | 5.1170 | | 4.8750 |
| 4 | 78.7927 | | 80.0260 | | 81.2054 | | 82.3314 | |
| | | 5.8424 | | 5.6330 | | 5.4239 | | 5.2162 |
| 5 | 72.9503 | | 74.3930 | | 75.7815 | | 77.1152 | |
| | | 6.0760 | | 5.9146 | | 5.7493 | | 5.5814 |
| 6 | 66.8743 | | 68.4784 | | 70.0322 | | 71.5338 | |
| | | 6.3192 | | 6.2103 | | 6.0944 | | 5.9722 |

## 15-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 7 | 60.5551 | 6.5719 | 62.2681 | 6.5209 | 63.9378 | 6.4601 | 65.5616 | 6.3901 |
| 8 | 53.9832 | 6.8348 | 55.7472 | 6.8468 | 57.4777 | 6.8476 | 59.1715 | 6.8375 |
| 9 | 47.1484 | 7.1081 | 48.9004 | 7.1892 | 50.6301 | 7.2584 | 52.3340 | 7.3160 |
| 10 | 40.0403 | 7.3925 | 41.7112 | 7.5487 | 43.3717 | 7.6941 | 45.0180 | 7.8282 |
| 11 | 32.6478 | 7.6882 | 34.1625 | 7.9261 | 35.6776 | 8.1555 | 37.1898 | 8.3762 |
| 12 | 24.9596 | 7.9957 | 26.2364 | 8.3224 | 27.5221 | 8.6450 | 28.8136 | 8.9625 |
| 13 | 16.9639 | 8.3156 | 17.9140 | 8.7385 | 18.8771 | 9.1636 | 19.8511 | 9.5899 |
| 14 | 8.6483 | 8.6483 | 9.1755 | 9.1755 | 9.7135 | 9.7135 | 10.2612 | 10.2612 |
| 15 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 20-Year Life.

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 0 | 100.0000 | 3.3582 | 100.0000 | 3.0243 | 100.0000 | 2.7185 | 100.0000 | 2.4393 |
| 1 | 96.6418 | 3.4925 | 96.9757 | 3.1755 | 97.2815 | 2.8815 | 97.5607 | 2.6100 |
| 2 | 93.1493 | 3.6322 | 93.8002 | 3.3342 | 94.4000 | 3.0545 | 94.9507 | 2.7928 |
| 3 | 89.5171 | 3.7775 | 90.4660 | 3.5010 | 91.3455 | 3.2377 | 92.1579 | 2.9882 |
| 4 | 85.7396 | 3.9286 | 86.9650 | 3.6760 | 88.1078 | 3.4320 | 89.1697 | 3.1974 |
| 5 | 81.8110 | 4.0857 | 83.2890 | 3.8598 | 84.6758 | 3.6379 | 85.9723 | 3.4213 |
| 6 | 77.7253 | 4.2492 | 79.4292 | 4.0528 | 81.0379 | 3.8561 | 82.5510 | 3.6607 |
| 7 | 73.4761 | 4.4191 | 75.3764 | 4.2554 | 77.1818 | 4.0876 | 78.8903 | 3.9169 |
| 8 | 69.0570 | 4.5959 | 71.1210 | 4.4682 | 73.0942 | 4.3328 | 74.9734 | 4.1912 |
| 9 | 64.4611 | 4.7797 | 66.6528 | 4.6916 | 68.7614 | 4.5928 | 70.7822 | 4.4845 |
| 10 | 59.6814 | 4.9709 | 61.9612 | 4.9262 | 64.1686 | 4.8684 | 66.2977 | 4.7985 |
| 11 | 54.7105 | 5.1698 | 57.0350 | 5.1725 | 59.3002 | 5.1604 | 61.4992 | 5.1343 |
| 12 | 49.5407 | 5.3766 | 51.8625 | 5.4311 | 54.1398 | 5.4701 | 56.3649 | 5.4938 |
| 13 | 44.1641 | 5.5916 | 46.4314 | 5.7027 | 48.6697 | 5.7982 | 50.8711 | 5.8783 |
| 14 | 38.5725 | 5.8152 | 40.7287 | 5.9878 | 42.8715 | 6.1462 | 44.9928 | 6.2898 |
| 15 | 32.7573 | 6.0479 | 34.7409 | 6.2872 | 36.7253 | 6.5149 | 38.7030 | 6.7301 |
| 16 | 26.7094 | 6.2898 | 28.4537 | 6.6016 | 30.2104 | 6.9059 | 31.9729 | 7.2012 |
| 17 | 20.4196 | 6.5414 | 21.8521 | 6.9317 | 23.3045 | 7.3202 | 24.7717 | 7.7052 |
| 18 | 13.8782 | 6.8031 | 14.9204 | 7.2783 | 15.9843 | 7.7593 | 17.0665 | 8.2446 |
| 19 | 7.0751 | 7.0741 | 7.6421 | 7.6421 | 8.2250 | 8.2250 | 8.8219 | 8.8219 |
| 20 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 25-Year Life.

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 0 | 100.0000 | 2.4012 | 100.0000 | 2.0952 | 100.0000 | 1.8227 | 100.0000 | 1.5811 |
| 1 | 97.5988 | 2.4972 | 97.9048 | 2.2001 | 98.1773 | 1.9320 | 98.4189 | 1.6917 |
| 2 | 95.1016 | 2.5972 | 95.7047 | 2.3100 | 96.2453 | 2.0480 | 96.7272 | 1.8101 |
| 3 | 92.5044 | 2.7010 | 93.3947 | 2.4254 | 94.1973 | 2.1708 | 94.9171 | 1.9369 |
| 4 | 89.8034 | 2.8091 | 90.9693 | 2.5468 | 92.0265 | 2.3011 | 92.9802 | 2.0724 |
| 5 | 86.9943 | 2.9214 | 88.4225 | 2.6742 | 89.7254 | 2.4391 | 90.9078 | 2.2175 |
| 6 | 84.0729 | 3.0383 | 85.7483 | 2.8078 | 87.2863 | 2.5855 | 88.6903 | 2.3728 |
| 7 | 81.0346 | 3.1598 | 82.9405 | 2.9482 | 84.7008 | 2.7406 | 86.3175 | 2.5388 |
| 8 | 77.8748 | 3.2862 | 79.9923 | 3.0957 | 81.9602 | 2.9051 | 83.7787 | 2.7165 |
| 9 | 74.5886 | 3.4176 | 76.8966 | 3.2504 | 79.0551 | 3.0794 | 81.0622 | 2.9067 |
| 10 | 71.1710 | 3.5544 | 73.6462 | 3.4129 | 75.9757 | 3.2641 | 78.1555 | 3.1102 |
| 11 | 67.6166 | 3.6965 | 70.2333 | 3.5836 | 72.7116 | 3.4600 | 75.0453 | 3.3279 |
| 12 | 63.9201 | 3.8444 | 66.6497 | 3.7627 | 69.2516 | 3.6675 | 71.7174 | 3.5608 |
| 13 | 60.0757 | 3.9982 | 62.8870 | 3.9509 | 65.5841 | 3.8877 | 68.1566 | 3.8101 |
| 14 | 56.0775 | 4.1581 | 58.9361 | 4.1485 | 61.6964 | 4.1208 | 64.3465 | 4.0768 |
| 15 | 51.9194 | 4.3244 | 54.7876 | 4.3559 | 57.5756 | 4.3682 | 60.2697 | 4.3622 |
| 16 | 47.5950 | 4.4974 | 50.4317 | 4.5736 | 53.2074 | 4.6302 | 55.9075 | 4.6675 |
| 17 | 43.0976 | 4.6773 | 45.8581 | 4.8024 | 48.5772 | 4.9081 | 51.2400 | 4.9942 |
| 18 | 38.4203 | 4.8643 | 41.0557 | 5.0424 | 43.6691 | 5.2025 | 46.2458 | 5.3439 |
| 19 | 33.5560 | 5.0590 | 36.0133 | 5.2946 | 38.4666 | 5.5147 | 40.9019 | 5.7179 |
| 20 | 28.4970 | 5.2613 | 30.7187 | 5.5593 | 32.9519 | 5.8455 | 35.1840 | 6.1181 |
| 21 | 23.2357 | 5.4718 | 25.1594 | 5.8373 | 27.1064 | 6.1963 | 29.0659 | 6.5465 |
| 22 | 17.7639 | 5.6906 | 19.3221 | 6.1291 | 20.9101 | 6.5681 | 22.5194 | 7.0047 |
| 23 | 12.0733 | 5.9183 | 13.1930 | 6.4356 | 14.3420 | 6.9621 | 15.5147 | 7.4950 |
| 24 | 6.1550 | 6.1550 | 6.7574 | 6.7574 | 7.3799 | 7.3799 | 8.0197 | 8.0197 |
| 25 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 30-Year Life.

| Age, in years. | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 1.7830 | 100.0000 | 1.5051 | 100.0000 | 1.2649 | 100.0000 | 1.0586 |
| 1 | 98.2170 | 1.8543 | 98.4949 | 1.5804 | 98.7351 | 1.3408 | 98.9414 | 1.1328 |
| 2 | 96.3627 | 1.9285 | 96.9145 | 1.6595 | 97.3943 | 1.4212 | 97.8086 | 1.2120 |
| 3 | 94.4342 | 2.0057 | 95.2550 | 1.7423 | 95.9731 | 1.5065 | 96.5966 | 1.2969 |

### 30-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 4 | 92.4285 | 2.0859 | 93.5127 | 1.8296 | 94.4666 | 1.5969 | 95.2997 | 1.3877 |
| 5 | 90.3426 | 2.1693 | 91.6831 | 1.9209 | 92.8697 | 1.6927 | 93.9120 | 1.4848 |
| 6 | 88.1733 | 2.2560 | 89.7622 | 2.0171 | 91.1770 | 1.7943 | 92.4272 | 1.5887 |
| 7 | 85.9173 | 2.3463 | 87.7451 | 2.1179 | 89.3827 | 1.9019 | 90.8385 | 1.6999 |
| 8 | 83.5710 | 2.4402 | 85.6272 | 2.2238 | 87.4808 | 2.0161 | 89.1386 | 1.8190 |
| 9 | 81.1308 | 2.5378 | 83.4034 | 2.3349 | 85.4647 | 2.1370 | 87.3196 | 1.9462 |
| 10 | 78.5930 | 2.6393 | 81.0685 | 2.4518 | 83.3277 | 2.2652 | 85.3734 | 2.0825 |
| 11 | 75.9537 | 2.7449 | 78.6167 | 2.5743 | 81.0625 | 2.4011 | 83.2909 | 2.2283 |
| 12 | 73.2088 | 2.8545 | 76.0424 | 2.7030 | 78.6614 | 2.5452 | 81.0626 | 2.3843 |
| 13 | 70.3543 | 2.9689 | 73.3394 | 2.8382 | 76.1162 | 2.6980 | 78.6783 | 2.5511 |
| 14 | 67.3854 | 3.0877 | 70.5012 | 2.9800 | 73.4182 | 2.8597 | 76.1272 | 2.7297 |
| 15 | 64.2977 | 3.2111 | 67.5212 | 3.1291 | 70.5585 | 3.0314 | 73.3975 | 2.9209 |
| 16 | 61.0866 | 3.3395 | 64.3921 | 3.2856 | 67.5271 | 3.2133 | 70.4766 | 3.1254 |
| 17 | 57.7471 | 3.4731 | 61.1065 | 3.4498 | 64.3138 | 3.4061 | 67.3512 | 3.3439 |
| 18 | 54.2740 | 3.6121 | 57.6567 | 3.6223 | 60.9077 | 3.6104 | 64.0073 | 3.5782 |
| 19 | 50.6619 | 3.7565 | 54.0344 | 3.8034 | 57.2973 | 3.8271 | 60.4291 | 3.8286 |
| 20 | 46.9054 | 3.9068 | 50.2310 | 3.9936 | 53.4702 | 4.0566 | 56.6005 | 4.0966 |
| 21 | 42.9986 | 4.0631 | 46.2374 | 4.1933 | 49.4136 | 4.3001 | 52.5039 | 4.3833 |
| 22 | 38.9355 | 4.2256 | 42.0441 | 4.4029 | 45.1135 | 4.5581 | 48.1206 | 4.6902 |
| 23 | 34.7099 | 4.3946 | 37.6412 | 4.6231 | 40.5554 | 4.8316 | 43.4304 | 5.0186 |
| 24 | 30.3153 | 4.5704 | 33.0181 | 4.8543 | 35.7238 | 5.1214 | 38.4118 | 5.3698 |
| 25 | 25.7449 | 4.7532 | 28.1638 | 5.0968 | 30.6024 | 5.4288 | 33.0420 | 5.7457 |
| 26 | 20.9917 | 4.9433 | 23.0670 | 5.3519 | 25.1736 | 5.7544 | 27.2963 | 6.1479 |
| 27 | 16.0484 | 5.1411 | 17.7151 | 5.6194 | 19.4192 | 6.0998 | 21.1484 | 6.5782 |
| 28 | 10.9073 | 5.3467 | 12.0957 | 5.9005 | 13.3194 | 6.4657 | 14.5702 | 7.0387 |
| 29 | 5.5606 | 5.5606 | 6.1952 | 6.1952 | 6.8537 | 6.8537 | 7.5315 | 7.5315 |
| 30 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

### 35-Year Life.

| Age, in years. | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 1.3577 | 100.0000 | 1.1072 | 100.0000 | 0.8974 | 100.0000 | 0.7234 |
| 1 | 98.6423 | 1.4121 | 98.8928 | 1.1625 | 99.1026 | 0.9512 | 99.2766 | 0.7740 |
| 2 | 97.2302 | 1.4685 | 97.7303 | 1.2207 | 98.1514 | 1.0083 | 98.5026 | 0.8282 |
| 3 | 95.7617 | 1.5273 | 96.5096 | 1.2816 | 97.1431 | 1.0688 | 97.6744 | 0.8862 |

35-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 4 | 94.2344 | 1.5884 | 95.2280 | 1.3458 | 96.0743 | 1.1329 | 96.7882 | 0.9483 |
| 5 | 92.6460 | 1.6519 | 93.8822 | 1.4131 | 94.9414 | 1.2010 | 95.8399 | 1.0146 |
| 6 | 90.9941 | 1.7179 | 92.4691 | 1.4837 | 93.7404 | 1.2729 | 94.8253 | 1.0856 |
| 7 | 89.2762 | 1.7867 | 90.9854 | 1.5579 | 92.4675 | 1.3493 | 93.7397 | 1.1616 |
| 8 | 87.4895 | 1.8581 | 89.4275 | 1.6358 | 91.1182 | 1.4303 | 92.5781 | 1.2429 |
| 9 | 85.6314 | 1.9325 | 87.7917 | 1.7176 | 89.6879 | 1.5162 | 91.3352 | 1.3300 |
| 10 | 83.6989 | 2.0098 | 86.0741 | 1.8035 | 88.1717 | 1.6071 | 90.0052 | 1.4230 |
| 11 | 81.6891 | 2.0902 | 84.2706 | 1.8936 | 86.5646 | 1.7034 | 88.5822 | 1.5226 |
| 12 | 79.5989 | 2.1737 | 82.3770 | 1.9883 | 84.8612 | 1.8058 | 87.0596 | 1.6293 |
| 13 | 77.4252 | 2.2607 | 80.3887 | 2.0877 | 83.0554 | 1.9140 | 85.4303 | 1.7432 |
| 14 | 75.1645 | 2.3512 | 78.3010 | 2.1922 | 81.1414 | 2.0289 | 83.6871 | 1.8653 |
| 15 | 72.8133 | 2.4452 | 76.1088 | 2.3017 | 79.1125 | 2.1506 | 81.8218 | 1.9959 |
| 16 | 70.3681 | 2.5430 | 73.8071 | 2.4168 | 76.9619 | 2.2797 | 79.8259 | 2.1356 |
| 17 | 67.8251 | 2.6447 | 71.3903 | 2.5377 | 74.6822 | 2.4165 | 77.6903 | 2.2851 |
| 18 | 65.1804 | 2.7505 | 68.8526 | 2.6645 | 72.2657 | 2.5614 | 75.4052 | 2.4450 |
| 19 | 62.4299 | 2.8605 | 66.1881 | 2.7978 | 69.7043 | 2.7152 | 72.9602 | 2.6162 |
| 20 | 59.5694 | 2.9750 | 63.3903 | 2.9376 | 66.9891 | 2.8780 | 70.3440 | 2.7993 |
| 21 | 56.5944 | 3.0939 | 60.4527 | 3.0845 | 64.1111 | 3.0507 | 67.5447 | 2.9953 |
| 22 | 53.5005 | 3.2178 | 57.3682 | 3.2388 | 61.0604 | 3.2338 | 64.5494 | 3.2049 |
| 23 | 50.2827 | 3.3464 | 54.1294 | 3.4007 | 57.8266 | 3.4278 | 61.3445 | 3.4293 |
| 24 | 46.9363 | 3.4803 | 50.7287 | 3.5708 | 54.3988 | 3.6334 | 57.9152 | 3.6693 |
| 25 | 43.4560 | 3.6195 | 47.1579 | 3.7492 | 50.7654 | 3.8515 | 54.2459 | 3.9262 |
| 26 | 39.8365 | 3.7642 | 43.4087 | 3.9367 | 46.9139 | 4.0826 | 50.3197 | 4.2010 |
| 27 | 36.0723 | 3.9149 | 39.4720 | 4.1337 | 42.8313 | 4.3275 | 46.1197 | 4.4951 |
| 28 | 32.1574 | 4.0714 | 35.3383 | 4.3403 | 38.5038 | 4.5871 | 41.6236 | 4.8097 |
| 29 | 28.0860 | 4.2343 | 30.9980 | 4.5572 | 33.9167 | 4.8624 | 36.8139 | 5.1464 |
| 30 | 23.8517 | 4.4037 | 26.4418 | 4.7851 | 29.0543 | 5.1541 | 31.6675 | 5.5067 |
| 31 | 19.4480 | 4.5798 | 21.6557 | 5.0244 | 23.9002 | 5.4634 | 26.1608 | 5.8922 |
| 32 | 14.8682 | 4.7630 | 16.6313 | 5.2756 | 18.4368 | 5.7911 | 20.2686 | 6.3046 |
| 33 | 10.1052 | 4.9535 | 11.3557 | 5.5394 | 12.6457 | 6.1387 | 13.9640 | 6.7459 |
| 34 | 5.1517 | 5.1517 | 5.8163 | 5.8163 | 6.5070 | 6.5070 | 7.2181 | 7.2181 |
| 35 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 40-Year Life.

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 0 | 100.0000 | 1.0524 | 100.0000 | 0.8278 | 100.0000 | 0.6462 | 100.0000 | 0.5009 |
| 1 | 98.9476 | 1.0944 | 99.1722 | 0.8692 | 99.3538 | 0.6849 | 99.4991 | 0.5360 |
| 2 | 97.8532 | 1.1382 | 98.3030 | 0.9127 | 98.6689 | 0.7260 | 98.9631 | 0.5735 |
| 3 | 96.7150 | 1.1838 | 97.3903 | 0.9583 | 97.9429 | 0.7696 | 98.3896 | 0.6136 |
| 4 | 95.5312 | 1.2311 | 96.4320 | 1.0062 | 97.1733 | 0.8157 | 97.7760 | 0.6566 |
| 5 | 94.3001 | 1.2803 | 95.4258 | 1.0565 | 96.3576 | 0.8647 | 97.1194 | 0.7026 |
| 6 | 93.0198 | 1.3316 | 94.3693 | 1.1093 | 95.4929 | 0.9166 | 96.4168 | 0.7517 |
| 7 | 91.6882 | 1.3848 | 93.2600 | 1.1648 | 94.5763 | 0.9716 | 95.6651 | 0.8044 |
| 8 | 90.3034 | 1.4402 | 92.0952 | 1.2230 | 93.6047 | 1.0299 | 94.8607 | 0.8606 |
| 9 | 88.8632 | 1.4978 | 90.8722 | 1.2842 | 92.5748 | 1.0916 | 94.0001 | 0.9210 |
| 10 | 87.3654 | 1.5577 | 89.5880 | 1.3484 | 91.4832 | 1.1572 | 93.0791 | 0.9853 |
| 11 | 85.8077 | 1.6201 | 88.2396 | 1.4158 | 90.3260 | 1.2266 | 92.0938 | 1.0544 |
| 12 | 84.1876 | 1.6848 | 86.8238 | 1.4866 | 89.0994 | 1.3002 | 91.0394 | 1.1281 |
| 13 | 82.5028 | 1.7523 | 85.3372 | 1.5610 | 87.7992 | 1.3782 | 89.9113 | 1.2071 |
| 14 | 80.7505 | 1.8223 | 83.7762 | 1.6390 | 86.4210 | 1.4609 | 88.7042 | 1.2917 |
| 15 | 78.9282 | 1.8952 | 82.1372 | 1.7210 | 84.9601 | 1.5485 | 87.4125 | 1.3820 |
| 16 | 77.0330 | 1.9710 | 80.4162 | 1.8070 | 83.4116 | 1.6415 | 86.0304 | 1.4788 |
| 17 | 75.0620 | 2.0499 | 78.6092 | 1.8974 | 81.7701 | 1.7399 | 84.5517 | 1.5823 |
| 18 | 73.0121 | 2.1319 | 76.7118 | 1.9923 | 80.0302 | 1.8443 | 82.9694 | 1.6930 |
| 19 | 70.8802 | 2.2171 | 74.7195 | 2.0919 | 78.1859 | 1.9550 | 81.2764 | 1.8116 |
| 20 | 68.6631 | 2.3059 | 72.6276 | 2.1965 | 76.2309 | 2.0724 | 79.4648 | 1.9384 |
| 21 | 66.3572 | 2.3980 | 70.4311 | 2.3063 | 74.1585 | 2.1966 | 77.5264 | 2.0741 |
| 22 | 63.9592 | 2.4940 | 68.1248 | 2.4216 | 71.9619 | 2.3284 | 75.4523 | 2.2192 |
| 23 | 61.4652 | 2.5937 | 65.7032 | 2.5427 | 69.6335 | 2.4682 | 73.2331 | 2.3746 |
| 24 | 58.8715 | 2.6975 | 63.1605 | 2.6698 | 67.1653 | 2.6162 | 70.8585 | 2.5408 |
| 25 | 56.1740 | 2.8054 | 60.4907 | 2.8033 | 64.5491 | 2.7732 | 68.3177 | 2.7187 |
| 26 | 53.3686 | 2.9176 | 57.6874 | 2.9434 | 61.7759 | 2.9396 | 65.5990 | 2.9090 |
| 27 | 50.4510 | 3.0343 | 54.7440 | 3.0906 | 58.8363 | 3.1160 | 62.6900 | 3.1126 |
| 28 | 47.4167 | 3.1557 | 51.6534 | 3.2451 | 55.7203 | 3.3029 | 59.5774 | 3.3305 |
| 29 | 44.2610 | 3.2819 | 48.4083 | 3.4074 | 52.4174 | 3.5011 | 56.2469 | 3.5636 |
| 30 | 40.9791 | 3.4132 | 45.0009 | 3.5778 | 48.9163 | 3.7112 | 52.6833 | 3.8131 |
| 31 | 37.5659 | 3.5498 | 41.4231 | 3.7567 | 45.2051 | 3.9339 | 48.8702 | 4.0800 |
| 32 | 34.0161 | 3.6916 | 37.6664 | 3.9445 | 41.2712 | 4.1699 | 44.7902 | 4.3656 |

### 40-Year Life.—(*Continued.*)

| Age in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 33 | 30.3245 | 3.8393 | 33.7219 | 4.1417 | 37.1013 | 4.4200 | 40.4246 | 4.6712 |
| 34 | 26.4852 | 3.9930 | 29.5802 | 4.3488 | 32.6813 | 4.6853 | 35.7534 | 4.9982 |
| 35 | 22.4922 | 4.1526 | 25.2314 | 4.5662 | 27.9960 | 4.9664 | 30.7552 | 5.3480 |
| 36 | 18.3396 | 4.3189 | 20.6652 | 4.7946 | 23.0296 | 5.2644 | 25.4072 | 5.7225 |
| 37 | 14.0207 | 4.4915 | 15.8706 | 5.0343 | 17.7652 | 5.5802 | 19.6847 | 6.1229 |
| 38 | 9.5292 | 4.6712 | 10.8363 | 5.2860 | 12.1850 | 5.9151 | 13.5618 | 6.5516 |
| 39 | 4.8580 | 4.8580 | 5.5503 | 5.5503 | 6.2699 | 6.2699 | 7.0102 | 7.0102 |
| 40 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

### 45-Year Life.

| Age in years. | Value. (4%) | Dep. (4%) | Value. (5%) | Dep. (5%) | Value. (6%) | Dep. (6%) | Value. (7%) | Dep. (7%) |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 0.8262 | 100.0000 | 0.6262 | 100.0000 | 0.4700 | 100.0000 | 0.3500 |
| 1 | 99.1738 | 0.8593 | 99.3738 | 0.6575 | 99.5300 | 0.4983 | 99.5600 | 0.3744 |
| 2 | 98.3145 | 0.8937 | 98.7163 | 0.6903 | 99.0317 | 0.5283 | 99.2756 | 0.4007 |
| 3 | 97.4208 | 0.9294 | 98.0260 | 0.7249 | 98.5034 | 0.5597 | 98.8749 | 0.4287 |
| 4 | 96.4914 | 0.9666 | 97.3011 | 0.7611 | 97.9437 | 0.5934 | 98.4462 | 0.4587 |
| 5 | 95.5248 | 1.0053 | 96.5400 | 0.7992 | 97.3503 | 0.6290 | 97.9875 | 0.4908 |
| 6 | 94.5195 | 1.0455 | 95.7408 | 0.8392 | 96.7213 | 0.6668 | 97.4967 | 0.5252 |
| 7 | 93.4740 | 1.0872 | 94.9016 | 0.8810 | 96.0545 | 0.7068 | 96.9715 | 0.5620 |
| 8 | 92.3868 | 1.1308 | 94.0206 | 0.9251 | 95.3477 | 0.7492 | 96.4095 | 0.6013 |
| 9 | 91.2560 | 1.1760 | 93.0955 | 0.9714 | 94.5985 | 0.7941 | 95.8082 | 0.6434 |
| 10 | 90.0800 | 1.2230 | 92.1241 | 1.0200 | 93.8044 | 0.8418 | 95.1648 | 0.6884 |
| 11 | 88.8570 | 1.2720 | 91.1041 | 1.0710 | 92.9626 | 0.8923 | 94.4764 | 0.7366 |
| 12 | 87.5850 | 1.3229 | 90.0311 | 1.1245 | 92.0703 | 0.9459 | 93.7398 | 0.7882 |
| 13 | 86.2621 | 1.3757 | 88.9066 | 1.1807 | 91.1244 | 1.0025 | 92.9516 | 0.8433 |
| 14 | 84.8864 | 1.4308 | 87.7279 | 1.2398 | 90.1219 | 1.0628 | 92.1083 | 0.9024 |
| 15 | 83.4556 | 1.4880 | 86.4881 | 1.3018 | 89.0591 | 1.1265 | 91.2059 | 0.9655 |
| 16 | 81.9676 | 1.5476 | 85.1863 | 1.3668 | 87.9326 | 1.1941 | 90.2404 | 1.0331 |
| 17 | 80.4200 | 1.6094 | 83.8195 | 1.4352 | 86.7385 | 1.2657 | 89.2073 | 1.1055 |
| 18 | 78.8106 | 1.6738 | 82.3843 | 1.5070 | 85.4728 | 1.3417 | 88.1018 | 1.1828 |
| 19 | 77.1368 | 1.7408 | 80.8773 | 1.5823 | 84.1311 | 1.4222 | 86.9190 | 1.2657 |
| 20 | 75.3960 | 1.8104 | 79.2950 | 1.6614 | 82.7089 | 1.5075 | 85.6533 | 1.3542 |
| 21 | 73.5856 | 1.8828 | 77.6336 | 1.7445 | 81.2014 | 1.5979 | 84.2991 | 1.4490 |

### 45-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 22 | 71.7028 | 1.9581 | 75.8891 | 1.8317 | 79.6035 | 1.6939 | 82.8501 | 1.5504 |
| 23 | 69.7447 | 2.0365 | 74.0574 | 1.9233 | 77.9096 | 1.7954 | 81.2997 | 1.6590 |
| 24 | 67.7082 | 2.1180 | 72.1341 | 2.0195 | 76.1142 | 1.9033 | 79.6407 | 1.7751 |
| 25 | 65.5902 | 2.2026 | 70.1146 | 2.1205 | 74.2109 | 2.0174 | 77.8656 | 1.8994 |
| 26 | 63.3876 | 2.2907 | 67.9941 | 2.2264 | 72.1935 | 2.1383 | 75.9662 | 2.0323 |
| 27 | 61.0969 | 2.3824 | 65.7677 | 2.3378 | 70.0552 | 2.2668 | 73.9339 | 2.1746 |
| 28 | 58.7145 | 2.4777 | 63.4299 | 2.4548 | 67.7884 | 2.4028 | 71.7593 | 2.3268 |
| 29 | 56.2368 | 2.5768 | 60.9751 | 2.5773 | 65.3856 | 2.5469 | 69.4325 | 2.4897 |
| 30 | 53.6600 | 2.6798 | 58.3978 | 2.7062 | 62.8387 | 2.6997 | 66.9428 | 2.6640 |
| 31 | 50.9802 | 2.7870 | 55.6916 | 2.8417 | 60.1390 | 2.8617 | 64.2788 | 2.8504 |
| 32 | 48.1932 | 2.8986 | 52.8499 | 2.9837 | 57.2773 | 3.0334 | 61.4284 | 3.0499 |
| 33 | 45.2946 | 3.0144 | 49.8662 | 3.1328 | 54.2439 | 3.2155 | 58.3785 | 3.2635 |
| 34 | 42.2802 | 3.1350 | 46.7334 | 3.2896 | 51.0284 | 3.4083 | 55.1150 | 3.4920 |
| 35 | 39.1452 | 3.2605 | 43.4438 | 3.4539 | 47.6201 | 3.6129 | 51.6230 | 3.7363 |
| 36 | 35.8847 | 3.3909 | 39.9899 | 3.6267 | 44.0072 | 3.8296 | 47.8867 | 3.9979 |
| 37 | 32.4938 | 3.5264 | 36.3632 | 3.8081 | 40.1776 | 4.0594 | 43.8888 | 4.2777 |
| 38 | 28.9674 | 3.6676 | 32.5551 | 3.9983 | 36.1182 | 4.3029 | 39.6111 | 4.5772 |
| 39 | 25.2998 | 3.8142 | 28.5568 | 4.1984 | 31.8153 | 4.5612 | 35.0339 | 4.8976 |
| 40 | 21.4856 | 3.9669 | 24.3584 | 4.4083 | 27.2541 | 4.8348 | 30.1363 | 5.2404 |
| 41 | 17.5187 | 4.1255 | 19.9501 | 4.6286 | 22.4193 | 5.1249 | 24.8959 | 5.6073 |
| 42 | 13.3932 | 4.2905 | 15.3215 | 4.8600 | 17.2944 | 5.4323 | 19.2886 | 5.9997 |
| 43 | 9.1027 | 4.4621 | 10.4615 | 5.1032 | 11.8621 | 5.7584 | 13.2889 | 6.4197 |
| 44 | 4.6406 | 4.6406 | 5.3583 | 5.3583 | 6.1037 | 6.1037 | 6.8692 | 6.8692 |
| 45 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

### 50-Year Life.

| Age, in years. | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 0.6550 | 100.0000 | 0.4777 | 100.0000 | 0.3444 | 100.0000 | 0.2460 |
| 1 | 99.3450 | 0.6812 | 99.5223 | 0.5015 | 99.6556 | 0.3651 | 99.7540 | 0.2632 |
| 2 | 98.6638 | 0.7085 | 99.0208 | 0.5267 | 99.2905 | 0.3870 | 99.4908 | 0.2816 |
| 3 | 97.9553 | 0.7368 | 98.4941 | 0.5529 | 98.9035 | 0.4102 | 99.2092 | 0.3014 |
| 4 | 97.2185 | 0.7663 | 97.9412 | 0.5806 | 98.4933 | 0.4349 | 98.9078 | 0.3224 |
| 5 | 96.4522 | 0.7969 | 97.3606 | 0.6097 | 98.0584 | 0.4609 | 98.5854 | 0.3450 |

50-YEAR LIFE.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 6 | 95.6553 | 0.8288 | 96.7509 | 0.6401 | 97.5975 | 0.4886 | 98.2404 | 0.3692 |
| 7 | 94.8265 | 0.8620 | 96.1108 | 0.6722 | 97.1089 | 0.5179 | 97.8712 | 0.3950 |
| 8 | 93.9645 | 0.8964 | 95.4386 | 0.7057 | 96.5910 | 0.5489 | 97.4762 | 0.4226 |
| 9 | 93.0681 | 0.9323 | 94.7329 | 0.7410 | 96.0421 | 0.5819 | 97.0536 | 0.4522 |
| 10 | 92.1358 | 0.9696 | 93.9919 | 0.7781 | 95.4602 | 0.6169 | 96.6014 | 0.4839 |
| 11 | 91.1662 | 1.0084 | 93.2138 | 0.8170 | 94.8433 | 0.6538 | 96.1175 | 0.5178 |
| 12 | 90.1578 | 1.0487 | 92.3968 | 0.8578 | 94.1895 | 0.6931 | 95.5997 | 0.5540 |
| 13 | 89.1091 | 1.0906 | 91.5390 | 0.9008 | 93.4964 | 0.7346 | 95.0457 | 0.5928 |
| 14 | 88.0185 | 1.1343 | 90.6382 | 0.9457 | 92.7618 | 0.7787 | 94.4529 | 0.6343 |
| 15 | 86.8842 | 1.1797 | 89.6925 | 0.9931 | 91.9831 | 0.8255 | 93.8186 | 0.6786 |
| 16 | 85.7045 | 1.2268 | 88.6994 | 1.0427 | 91.1576 | 0.8749 | 93.1400 | 0.7262 |
| 17 | 84.4777 | 1.2759 | 87.6567 | 1.0948 | 90.2827 | 0.9275 | 92.4138 | 0.7771 |
| 18 | 83.2018 | 1.3270 | 86.5619 | 1.1496 | 89.3552 | 0.9831 | 91.6367 | 0.8314 |
| 19 | 81.8748 | 1.3800 | 85.4123 | 1.2071 | 88.3721 | 1.0421 | 90.8053 | 0.8896 |
| 20 | 80.4948 | 1.4353 | 84.2052 | 1.2674 | 87.3300 | 1.1046 | 89.9157 | 0.9519 |
| 21 | 79.0595 | 1.4926 | 82.9378 | 1.3307 | 86.2254 | 1.1710 | 88.9638 | 1.0185 |
| 22 | 77.5669 | 1.5523 | 81.6017 | 1.3974 | 85.0544 | 1.2411 | 87.9453 | 1.0898 |
| 23 | 76.0146 | 1.6144 | 80.2097 | 1.4672 | 83.8133 | 1.3156 | 86.8555 | 1.1661 |
| 24 | 74.4002 | 1.6791 | 78.7425 | 1.5405 | 82.4977 | 1.3946 | 85.6894 | 1.2477 |
| 25 | 72.7211 | 1.7461 | 77.2020 | 1.6176 | 81.1031 | 1.4782 | 84.4417 | 1.3351 |
| 26 | 70.9750 | 1.8161 | 75.5844 | 1.6984 | 79.6249 | 1.5670 | 83.1066 | 1.4285 |
| 27 | 69.1589 | 1.8887 | 73.8860 | 1.7834 | 78.0579 | 1.6610 | 81.6781 | 1.5285 |
| 28 | 67.2702 | 1.9642 | 72.1026 | 1.8726 | 76.3969 | 1.7606 | 80.1496 | 1.6355 |
| 29 | 65.3060 | 2.0427 | 70.2300 | 1.9661 | 74.6363 | 1.8662 | 78.5141 | 1.7500 |
| 30 | 63.2633 | 2.1245 | 68.2639 | 2.0645 | 72.7701 | 1.9783 | 76.7641 | 1.8725 |
| 31 | 61.1388 | 2.2095 | 66.1994 | 2.1677 | 70.7918 | 2.0969 | 74.8916 | 2.0036 |
| 32 | 58.9293 | 2.2979 | 64.0317 | 2.2761 | 68.6949 | 2.2227 | 72.8880 | 2.1438 |
| 33 | 56.6314 | 2.3897 | 61.7556 | 2.3899 | 66.4722 | 2.3561 | 70.7442 | 2.2939 |
| 34 | 54.2417 | 2.4854 | 59.3657 | 2.5095 | 64.1161 | 2.4975 | 68.4503 | 2.4545 |
| 35 | 51.7563 | 2.5847 | 56.8562 | 2.6347 | 61.6186 | 2.6473 | 65.9958 | 2.6263 |
| 36 | 49.1716 | 2.6882 | 54.2215 | 2.7666 | 58.9713 | 2.8061 | 63.3695 | 2.8101 |
| 37 | 46.4834 | 2.7957 | 51.4549 | 2.9050 | 56.1652 | 2.9745 | 60.5594 | 3.0068 |
| 38 | 43.6877 | 2.9075 | 48.5499 | 3.0501 | 53.1907 | 3.1530 | 57.5526 | 3.2173 |

## 50-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 39 | 40.7802 | 3.0238 | 45.4998 | 3.2027 | 50.0377 | 3.3422 | 54.3353 | 3.4425 |
| 40 | 37.7564 | 3.1448 | 42.2971 | 3.3629 | 46.6955 | 3.5427 | 50.8928 | 3.6835 |
| 41 | 34.6116 | 3.2706 | 38.9342 | 3.5309 | 43.1528 | 3.7553 | 47.2093 | 3.9414 |
| 42 | 31.3410 | 3.4013 | 35.4033 | 3.7075 | 39.3975 | 3.9805 | 43.2679 | 4.2172 |
| 43 | 27.9397 | 3.5375 | 31.6958 | 3.8929 | 35.4170 | 4.2194 | 39.0507 | 4.5124 |
| 44 | 24.4022 | 3.6789 | 27.8029 | 4.0875 | 31.1976 | 4.4726 | 34.5383 | 4.8284 |
| 45 | 20.7233 | 3.8261 | 23.7154 | 4.2919 | 26.7250 | 4.7410 | 29.7099 | 5.1662 |
| 46 | 16.8972 | 3.9791 | 19.4235 | 4.5065 | 21.9840 | 5.0253 | 24.5437 | 5.5280 |
| 47 | 12.9181 | 4.1383 | 14.9170 | 4.7319 | 16.9587 | 5.3270 | 19.0157 | 5.9149 |
| 48 | 8.7798 | 4.3038 | 10.1851 | 4.9683 | 11.6317 | 5.6465 | 13.1008 | 6.3289 |
| 49 | 4.4760 | 4.4760 | 5.2168 | 5.2168 | 5.9852 | 5.9852 | 6.7719 | 6.7719 |
| 50 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 60-Year Life.

| Age, in years. | Value. 4% | Dep. 4% | Value. 5% | Dep. 5% | Value. 6% | Dep. 6% | Value. 7% | Dep. 7% |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 0.4202 | 100.0000 | 0.2828 | 100.0000 | 0.1876 | 100.0000 | 0.1229 |
| 1 | 99.5798 | 0.4370 | 99.9172 | 0.2970 | 99.8124 | 0.1988 | 99.8771 | 0.1316 |
| 2 | 99.1428 | 0.4544 | 99.4202 | 0.3118 | 99.6136 | 0.2108 | 99.7455 | 0.1407 |
| 3 | 98.6884 | 0.4727 | 99.1084 | 0.3274 | 99.4028 | 0.2234 | 99.6048 | 0.1506 |
| 4 | 98.2157 | 0.4916 | 98.7810 | 0.3437 | 99.1794 | 0.2368 | 99.4542 | 0.1611 |
| 5 | 97.7241 | 0.5112 | 98.4373 | 0.3610 | 98.9426 | 0.2510 | 99.2931 | 0.1724 |
| 6 | 97.2129 | 0.5317 | 98.0763 | 0.3790 | 98.6916 | 0.2661 | 99.1207 | 0.1845 |
| 7 | 96.6812 | 0.5529 | 97.6973 | 0.3980 | 98.4255 | 0.2820 | 98.9362 | 0.1974 |
| 8 | 96.1283 | 0.5750 | 97.2993 | 0.4178 | 98.1435 | 0.2989 | 98.7388 | 0.2112 |
| 9 | 95.5533 | 0.5981 | 96.8815 | 0.4388 | 97.8446 | 0.3169 | 98.5276 | 0.2660 |
| 10 | 94.9552 | 0.6220 | 96.4427 | 0.4606 | 97.5277 | 0.3360 | 98.3016 | 0.2418 |
| 11 | 94.3332 | 0.6468 | 95.9821 | 0.4837 | 97.1917 | 0.3560 | 98.0598 | 0.2587 |
| 12 | 93.6864 | 0.6728 | 95.4984 | 0.5079 | 96.8357 | 0.3775 | 97.8011 | 0.2768 |
| 13 | 93.0136 | 0.6996 | 94.9905 | 0.5333 | 96.4582 | 0.4000 | 97.5243 | 0.2963 |
| 14 | 92.3140 | 0.7276 | 94.4572 | 0.5600 | 96.0582 | 0.4241 | 97.2280 | 0.3169 |
| 15 | 91.5864 | 0.7567 | 93.8972 | 0.5880 | 95.6341 | 0.4495 | 96.9111 | 0.3392 |
| 16 | 90.8297 | 0.7870 | 93.3092 | 0.6174 | 95.1846 | 0.4765 | 96.5719 | 0.3629 |
| 17 | 90.0427 | 0.8185 | 92.6918 | 0.6481 | 94.7081 | 0.5051 | 96.2090 | 0.3883 |

## 60-Year Life.—(*Continued*).

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 18 | 89.2242 | 0.8512 | 92.0437 | 0.6807 | 94.2030 | 0.5354 | 95.8207 | 0.4154 |
| 19 | 88.3730 | 0.8853 | 91.3630 | 0.7146 | 93.6671 | 0.5675 | 95.4053 | 0.4446 |
| 20 | 87.4877 | 0.9207 | 90.6484 | 0.7504 | 93.1001 | 0.6016 | 94.9607 | 0.4757 |
| 21 | 86.5670 | 0.9575 | 89.8980 | 0.7880 | 92.4985 | 0.6377 | 94.4850 | 0.5089 |
| 22 | 85.6095 | 0.9958 | 89.1100 | 0.8273 | 91.8608 | 0.6759 | 93.9761 | 0.5446 |
| 23 | 84.6137 | 1.0356 | 88.2827 | 0.8686 | 91.1849 | 0.7165 | 93.4315 | 0.5828 |
| 24 | 83.5781 | 1.0771 | 87.4141 | 0.9122 | 90.4684 | 0.7594 | 92.8487 | 0.6235 |
| 25 | 82.5010 | 1.1201 | 86.5019 | 0.9577 | 89.7090 | 0.8051 | 92.2252 | 0.6671 |
| 26 | 81.3809 | 1.1650 | 85.5442 | 1.0056 | 88.9039 | 0.8533 | 91.5581 | 0.7139 |
| 27 | 80.2159 | 1.2115 | 84.5386 | 1.0559 | 88.0506 | 0.9045 | 90.8442 | 0.7638 |
| 28 | 79.0044 | 1.2600 | 83.4827 | 1.1087 | 87.1461 | 0.9589 | 90.0804 | 0.8173 |
| 29 | 77.7444 | 1.3104 | 82.3740 | 1.1641 | 86.1872 | 1.0163 | 89.2631 | 0.8745 |
| 30 | 76.4340 | 1.3629 | 81.2099 | 1.2223 | 85.1709 | 1.0773 | 88.3886 | 0.9357 |
| 31 | 75.0711 | 1.4173 | 79.9876 | 1.2835 | 84.0936 | 1.1420 | 87.4529 | 1.0012 |
| 32 | 73.6538 | 1.4741 | 78.7041 | 1.3476 | 82.9516 | 1.2104 | 86.4517 | 1.0714 |
| 33 | 72.1797 | 1.5329 | 77.3565 | 1.4149 | 81.7412 | 1.2831 | 85.3803 | 1.1462 |
| 34 | 70.6468 | 1.5944 | 75.9416 | 1.4858 | 80.4581 | 1.3601 | 84.2341 | 1.2266 |
| 35 | 69.0524 | 1.6580 | 74.4558 | 1.5600 | 79.0980 | 1.4417 | 83.0075 | 1.3124 |
| 36 | 67.3944 | 1.7245 | 72.8958 | 1.6380 | 77.6563 | 1.5282 | 81.6951 | 1.4043 |
| 37 | 65.6699 | 1.7934 | 71.2578 | 1.7200 | 76.1281 | 1.6199 | 80.2908 | 1.5025 |
| 38 | 63.8765 | 1.8651 | 69.5378 | 1.8059 | 74.5082 | 1.7171 | 78.7883 | 1.6078 |
| 39 | 62.0114 | 1.9397 | 67.7319 | 1.8962 | 72.7911 | 1.8201 | 77.1805 | 1.7208 |
| 40 | 60.0717 | 2.0173 | 65.8357 | 1.9911 | 70.9710 | 1.9293 | 75.4602 | 1.8407 |
| 41 | 58.0544 | 2.0980 | 63.8446 | 2.0906 | 69.0417 | 2.0452 | 73.6195 | 1.9695 |
| 42 | 55.9564 | 2.1820 | 61.7540 | 2.1951 | 66.9965 | 2.1677 | 71.6500 | 2.1074 |
| 43 | 53.7744 | 2.2692 | 59.5589 | 2.3049 | 64.8288 | 2.2978 | 69.5426 | 2.2550 |
| 44 | 51.5052 | 2.3599 | 57.2540 | 2.4201 | 62.5310 | 2.4357 | 67.2876 | 2.4128 |
| 45 | 49.1453 | 2.4544 | 54.8339 | 2.5411 | 60.0953 | 2.5819 | 64.8748 | 2.5816 |
| 46 | 46.6909 | 2.5526 | 52.2928 | 2.6682 | 57.5134 | 2.7367 | 62.2932 | 2.7625 |
| 47 | 44.1383 | 2.6546 | 49.6246 | 2.8016 | 54.7767 | 2.9010 | 59.5307 | 2.9557 |
| 48 | 41.4837 | 2.7609 | 46.8230 | 2.9416 | 51.8757 | 3.0750 | 56.5750 | 3.1627 |
| 49 | 38.7228 | 2.8712 | 43.8814 | 3.0888 | 48.8007 | 3.2596 | 53.4123 | 3.3841 |
| 50 | 35.8516 | 2.9861 | 40.7926 | 3.2432 | 45.5411 | 3.4551 | 50.0282 | 3.6209 |

## 60-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 51 | 32.8655 | 3.1056 | 37.5494 | 3.4053 | 42.0860 | 3.6624 | 46.4073 | 3.8745 |
| 52 | 29.7599 | 3.2298 | 34.1441 | 3.5756 | 38.4236 | 3.8821 | 42.5328 | 4.1456 |
| 53 | 26.5301 | 3.3590 | 30.5685 | 3.7544 | 34.5415 | 4.1151 | 38.3872 | 4.4358 |
| 54 | 23.1711 | 3.4933 | 26.8141 | 3.9421 | 30.4264 | 4.3620 | 33.9514 | 4.7463 |
| 55 | 19.6778 | 3.6331 | 22.8720 | 4.1392 | 26.0644 | 4.6237 | 29.2051 | 5.0785 |
| 56 | 16.0447 | 3.7784 | 18.7328 | 4.3462 | 21.4407 | 4.9012 | 24.1266 | 5.4341 |
| 57 | 12.2663 | 3.9295 | 14.3866 | 4.5635 | 16.5395 | 5.1952 | 18.6925 | 5.8145 |
| 58 | 8.3368 | 4.0867 | 9.8231 | 4.7917 | 11.3443 | 5.5069 | 12.8780 | 6.2214 |
| 59 | 4.2501 | 4.2501 | 5.0314 | 5.0314 | 5.8374 | 5.8374 | 6.6566 | 6.6566 |
| 60 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 80-Year Life.

| Age, in years. | 4% Value. | 4% Dep. | 5% Value. | 5% Dep. | 6% Value. | 6% Dep. | 7% Value. | 7% Dep. |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 0.1814 | 100.0000 | 0.1030 | 100.0000 | 0.0573 | 100.0000 | 0.0314 |
| 1 | 99.8186 | 0.1887 | 99.8970 | 0.1081 | 99.9427 | 0.0607 | 99.9686 | 0.0336 |
| 2 | 99.6299 | 0.1962 | 99.7889 | 0.1135 | 99.8820 | 0.0643 | 99.9350 | 0.0359 |
| 3 | 99.4337 | 0.2041 | 99.6754 | 0.1192 | 99.8177 | 0.0682 | 99.8991 | 0.0384 |
| 4 | 99.2296 | 0.2122 | 99.5562 | 0.1252 | 99.7495 | 0.0723 | 99.8607 | 0.0411 |
| 5 | 99.0174 | 0.2207 | 99.4310 | 0.1314 | 99.6772 | 0.0766 | 99.8196 | 0.0440 |
| 6 | 98.7967 | 0.2295 | 99.2996 | 0.1380 | 99.6006 | 0.0812 | 99.7756 | 0.0471 |
| 7 | 98.5672 | 0.2387 | 99.1616 | 0.1449 | 99.5194 | 0.0861 | 99.7285 | 0.0504 |
| 8 | 98.3285 | 0.2483 | 99.0167 | 0.1521 | 99.4333 | 0.0913 | 99.6781 | 0.0539 |
| 9 | 98.0802 | 0.2582 | 98.8646 | 0.1597 | 99.3420 | 0.0967 | 99.6242 | 0.0576 |
| 10 | 97.8220 | 0.2685 | 98.7049 | 0.1677 | 99.2453 | 0.1025 | 99.5666 | 0.0617 |
| 11 | 97.5535 | 0.2793 | 98.5372 | 0.1761 | 99.1428 | 0.1087 | 99.5049 | 0.0660 |
| 12 | 97.2742 | 0.2904 | 98.3611 | 0.1849 | 99.0341 | 0.1152 | 99.4389 | 0.0706 |
| 13 | 96.9838 | 0.3021 | 98.1762 | 0.1941 | 98.9189 | 0.1221 | 99.3683 | 0.0756 |
| 14 | 96.6817 | 0.3141 | 97.9821 | 0.2039 | 98.7968 | 0.1294 | 99.2927 | 0.0809 |
| 15 | 96.3676 | 0.3267 | 97.7782 | 0.2140 | 98.6674 | 0.1372 | 99.2118 | 0.0865 |
| 16 | 96.0409 | 0.3398 | 97.5642 | 0.2248 | 98.5302 | 0.1454 | 99.1253 | 0.0926 |
| 17 | 95.7011 | 0.3534 | 97.3394 | 0.2360 | 98.3848 | 0.1542 | 99.0327 | 0.0990 |
| 18 | 95.3477 | 0.3675 | 97.1034 | 0.2478 | 98.2306 | 0.1634 | 98.9337 | 0.1060 |

80-YEAR LIFE.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value | Dep. | Value. | Dep. |
| 19 | 94.9802 | 0.3822 | 96.8556 | 0.2602 | 98.0672 | 0.1732 | 98.8277 | 0.1134 |
| 20 | 94.5980 | 0.3975 | 96.5954 | 0.2732 | 97.8940 | 0.1836 | 98.7143 | 0.1213 |
| 21 | 94.2005 | 0.4134 | 96.3222 | 0.2869 | 97.7104 | 0.1946 | 98.5930 | 0.1298 |
| 22 | 93.7871 | 0.4299 | 96.0353 | 0.3012 | 97.5158 | 0.2063 | 98.4632 | 0.1389 |
| 23 | 93.3572 | 0.4471 | 95.7341 | 0.3162 | 97.3095 | 0.2187 | 98.3243 | 0.1485 |
| 24 | 92.9101 | 0.4650 | 95.4179 | 0.3321 | 97.0908 | 0.2318 | 98.1758 | 0.1590 |
| 25 | 92.4451 | 0.4836 | 95.0858 | 0.3487 | 96.8590 | 0.2457 | 98.0168 | 0.1703 |
| 26 | 91.9615 | 0.5030 | 94.7371 | 0.3661 | 96.6133 | 0.2605 | 97.8466 | 0.1821 |
| 27 | 91.4585 | 0.5231 | 94.3710 | 0.3844 | 96.3528 | 0.2761 | 97.6645 | 0.1948 |
| 28 | 90.9354 | 0.5440 | 93.9866 | 0.4036 | 96.0767 | 0.2927 | 97.4697 | 0.2085 |
| 29 | 90.3914 | 0.5658 | 93.5830 | 0.4238 | 95.7840 | 0.3102 | 97.2612 | 0.2231 |
| 30 | 89.8256 | 0.5884 | 93.1592 | 0.4450 | 95.4738 | 0.3288 | 97.0381 | 0.2387 |
| 31 | 89.2372 | 0.6119 | 92.7142 | 0.4672 | 95.1450 | 0.3486 | 96.7994 | 0.2554 |
| 32 | 88.6253 | 0.6364 | 92.2470 | 0.4906 | 94.7964 | 0.3695 | 96.5440 | 0.2733 |
| 33 | 87.9889 | 0.6618 | 91.7564 | 0.5151 | 94.4269 | 0.3917 | 96.2707 | 0.2924 |
| 34 | 87.3271 | 0.6883 | 91.2413 | 0.5409 | 94.0352 | 0.4152 | 95.9783 | 0.3129 |
| 35 | 86.6388 | 0.7159 | 90.7004 | 0.5679 | 93.6200 | 0.4401 | 95.6654 | 0.3348 |
| 36 | 85.9229 | 0.7445 | 90.1325 | 0.5963 | 93.1799 | 0.4665 | 95.3306 | 0.3582 |
| 37 | 85.1784 | 0.7743 | 89.5362 | 0.6262 | 92.7134 | 0.4945 | 94.9724 | 0.3833 |
| 38 | 84.4041 | 0.8052 | 88.9100 | 0.6575 | 92.2189 | 0.5241 | 94.5891 | 0.4101 |
| 39 | 83.5989 | 0.8375 | 88.2525 | 0.6903 | 91.6948 | 0.5556 | 94.1790 | 0.4388 |
| 40 | 82.7614 | 0.8709 | 87.5622 | 0.7249 | 91.1392 | 0.5889 | 93.7402 | 0.4696 |
| 41 | 81.8905 | 0.9058 | 86.8373 | 0.7611 | 90.5503 | 0.6242 | 93.2706 | 0.5024 |
| 42 | 80.9847 | 0.9420 | 86.0762 | 0.7992 | 89.9261 | 0.6617 | 92.7682 | 0.5376 |
| 43 | 80.0427 | 0.9797 | 85.2770 | 0.8391 | 89.2644 | 0.7014 | 92.2306 | 0.5752 |
| 44 | 79.0630 | 1.0189 | 84.4379 | 0.8811 | 88.5630 | 0.7435 | 91.6554 | 0.6155 |
| 45 | 78.0441 | 1.0596 | 83.5568 | 0.9251 | 87.8195 | 0.7881 | 91.0399 | 0.6586 |
| 46 | 76.9845 | 1.1020 | 82.6317 | 0.9714 | 87.0314 | 0.8354 | 90.3813 | 0.7047 |
| 47 | 75.8825 | 1.1461 | 81.6603 | 1.0199 | 86.1960 | 0.8855 | 89.6766 | 0.7540 |
| 48 | 74.7364 | 1.1919 | 80.6404 | 1.0709 | 85.3105 | 0.9386 | 88.9226 | 0.8068 |
| 49 | 73.5445 | 1.2396 | 79.5695 | 1.1245 | 84.3719 | 0.9949 | 88.1158 | 0.8633 |
| 50 | 72.3049 | 1.2892 | 78.4450 | 1.1807 | 83.3770 | 1.0546 | 87.2525 | 0.9237 |
| 51 | 71.0157 | 1.3408 | 77.2643 | 1.2397 | 82.3224 | 1.1179 | 86.3288 | 0.9883 |

## 80-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 52 | 69.6749 | 1.3944 | 76.0246 | 1.3017 | 81.2045 | 1.1850 | 85.3405 | 1.0575 |
| 53 | 68.2805 | 1.4502 | 74.7229 | 1.3668 | 80.0195 | 1.2561 | 84.2830 | 1.1316 |
| 54 | 66.8303 | 1.5082 | 73.3561 | 1.4352 | 78.7634 | 1.3314 | 83.1514 | 1.2108 |
| 55 | 65.3221 | 1.5685 | 71.9209 | 1.5069 | 77.4320 | 1.4113 | 81.9406 | 1.2955 |
| 56 | 63.7536 | 1.6312 | 70.4140 | 1.5823 | 76.0207 | 1.4960 | 80.6451 | 1.3862 |
| 57 | 62.1224 | 1.6965 | 68.8317 | 1.6614 | 74.5247 | 1.5858 | 79.2589 | 1.4832 |
| 58 | 60.4259 | 1.7644 | 67.1703 | 1.7445 | 72.9389 | 1.6809 | 77.7757 | 1.5871 |
| 59 | 58.6615 | 1.8349 | 65.4258 | 1.8317 | 71.2580 | 1.7818 | 76.1886 | 1.6982 |
| 60 | 56.8266 | 1.9083 | 63.5941 | 1.9233 | 69.4762 | 1.8887 | 74.4904 | 1.8170 |
| 61 | 54.9183 | 1.9847 | 61.6708 | 2.0194 | 67.5875 | 2.0020 | 72.6734 | 1.9442 |
| 62 | 52.9336 | 2.0640 | 59.6514 | 2.1204 | 65.5855 | 2.1221 | 70.7292 | 2.0803 |
| 63 | 50.8696 | 2.1466 | 57.5310 | 2.2264 | 63.4634 | 2.2495 | 68.6489 | 2.2259 |
| 64 | 48.7230 | 2.2325 | 55.3046 | 2.3377 | 61.2139 | 2.3844 | 66.4230 | 2.3818 |
| 65 | 46.4905 | 2.3218 | 52.9669 | 2.4546 | 58.8295 | 2.5275 | 64.0412 | 2.5485 |
| 66 | 44.1687 | 2.4147 | 50.5123 | 2.5773 | 56.3020 | 2.6791 | 61.4927 | 2.7269 |
| 67 | 41.7540 | 2.5112 | 47.9350 | 2.7062 | 53.6229 | 2.8399 | 58.7658 | 2.9178 |
| 68 | 39.2428 | 2.6117 | 45.2288 | 2.8415 | 50.7830 | 3.0103 | 55.8480 | 3.1220 |
| 69 | 36.6311 | 2.7162 | 42.3873 | 2.9836 | 47.7727 | 3.1909 | 52.7260 | 3.3405 |
| 70 | 33.9149 | 2.8248 | 39.4037 | 3.1328 | 44.5818 | 3.3823 | 49.3855 | 3.5744 |
| 71 | 31.0901 | 2.9378 | 36.2709 | 3.2894 | 41.1995 | 3.5853 | 45.8111 | 3.8246 |
| 72 | 28.1523 | 3.0553 | 32.9815 | 3.4539 | 37.6142 | 3.8004 | 41.9865 | 4.0923 |
| 73 | 25.0970 | 3.1775 | 29.5276 | 3.6266 | 33.8138 | 4.0284 | 37.8942 | 4.3788 |
| 74 | 21.9195 | 3.3046 | 25.9010 | 3.8079 | 29.7854 | 4.2701 | 33.5154 | 4.6853 |
| 75 | 18.6149 | 3.4368 | 22.0931 | 3.9983 | 25.5153 | 4.5263 | 28.8301 | 5.0133 |
| 76 | 15.1781 | 3.5743 | 18.0948 | 4.1982 | 20.9890 | 4.7979 | 23.8168 | 5.3642 |
| 77 | 11.6038 | 3.7173 | 13.8966 | 4.4081 | 16.1911 | 5.0858 | 18.4526 | 5.7397 |
| 78 | 7.8865 | 3.8659 | 9.4885 | 4.6285 | 11.1053 | 5.3909 | 12.7129 | 6.1415 |
| 79 | 4.0206 | 4.0206 | 4.8600 | 4.8600 | 5.7144 | 5.7144 | 6.5714 | 6.5714 |
| 80 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

## 100-Year Life.

| Age, in years. | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
|---|---|---|---|---|---|---|---|---|
| 0 | 100.0000 | 0.0808 | 100.0000 | 0.0383 | 100.0000 | 0.0177 | 100.0000 | 0.0081 |
| 1 | 99.9192 | 0.0840 | 99.9617 | 0.0402 | 99.9823 | 0.0188 | 99.9919 | 0.0086 |
| 2 | 99.8352 | 0.0874 | 99.9215 | 0.0423 | 99.9635 | 0.0199 | 99.9833 | 0.0092 |

100-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 3 | 99.7478 | 0.0909 | 99.8792 | 0.0444 | 99.9436 | 0.0211 | 99.9741 | 0.0099 |
| 4 | 99.6569 | 0.0945 | 99.8348 | 0.0466 | 99.9225 | 0.0224 | 99.9642 | 0.0106 |
| 5 | 99.5624 | 0.0983 | 99.7882 | 0.0489 | 99.9001 | 0.0237 | 99.9536 | 0.0113 |
| 6 | 99.4641 | 0.1022 | 99.7393 | 0.0514 | 99.8764 | 0.0251 | 99.9423 | 0.0121 |
| 7 | 99.3619 | 0.1063 | 99.6879 | 0.0539 | 99.8513 | 0.0267 | 99.9302 | 0.0130 |
| 8 | 99.2556 | 0.1106 | 99.6340 | 0.0566 | 99.8246 | 0.0283 | 99.9172 | 0.0139 |
| 9 | 99.1450 | 0.1150 | 99.5774 | 0.0594 | 99.7963 | 0.0300 | 99.9033 | 0.0148 |
| 10 | 99.0300 | 0.1196 | 99.5180 | 0.0624 | 99.7663 | 0.0318 | 99.8885 | 0.0159 |
| 11 | 98.9104 | 0.1244 | 99.4556 | 0.0655 | 99.7345 | 0.0337 | 99.8726 | 0.0170 |
| 12 | 98.7860 | 0.1294 | 99.3901 | 0.0688 | 99.7008 | 0.0357 | 99.8556 | 0.0182 |
| 13 | 98.6566 | 0.1345 | 99.3213 | 0.0723 | 99.6651 | 0.0378 | 99.8374 | 0.0195 |
| 14 | 98.5221 | 0.1399 | 99.2490 | 0.0759 | 99.6273 | 0.0401 | 99.8179 | 0.0208 |
| 15 | 98.3822 | 0.1455 | 99.1731 | 0.0797 | 99.5872 | 0.0425 | 99.7971 | 0.0223 |
| 16 | 98.2367 | 0.1513 | 99.0934 | 0.0836 | 99.5447 | 0.0450 | 99.7748 | 0.0238 |
| 17 | 98.0854 | 0.1574 | 99.0098 | 0.0878 | 99.4997 | 0.0477 | 99.7510 | 0.0255 |
| 18 | 97.9280 | 0.1637 | 98.9220 | 0.0922 | 99.4520 | 0.0506 | 99.7255 | 0.0273 |
| 19 | 97.7643 | 0.1702 | 98.8298 | 0.0968 | 99.4014 | 0.0537 | 99.6982 | 0.0292 |
| 20 | 97.5941 | 0.1770 | 98.7330 | 0.1017 | 99.3477 | 0.0569 | 99.6690 | 0.0313 |
| 21 | 97.4171 | 0.1841 | 98.6313 | 0.1067 | 99.2908 | 0.0603 | 99.6377 | 0.0334 |
| 22 | 97.2330 | 0.1915 | 98.5246 | 0.1121 | 99.2305 | 0.0639 | 99.6043 | 0.0358 |
| 23 | 97.0415 | 0.1992 | 98.4125 | 0.1177 | 99.1666 | 0.0677 | 99.5685 | 0.0383 |
| 24 | 96.8423 | 0.2071 | 98.2948 | 0.1236 | 99.0989 | 0.0718 | 99.5302 | 0.0410 |
| 25 | 96.6352 | 0.2154 | 98.1712 | 0.1298 | 99.0271 | 0.0761 | 99.4892 | 0.0438 |
| 26 | 96.4198 | 0.2240 | 98.0414 | 0.1362 | 98.9510 | 0.0807 | 99.4454 | 0.0469 |
| 27 | 96.1958 | 0.2330 | 97.9052 | 0.1431 | 98.8703 | 0.0855 | 99.3985 | 0.0502 |
| 28 | 95.9628 | 0.2423 | 97.7621 | 0.1502 | 98.7848 | 0.0906 | 99.3483 | 0.0537 |
| 29 | 95.7205 | 0.2520 | 97.6119 | 0.1577 | 98.6942 | 0.0961 | 99.2946 | 0.0575 |
| 30 | 95.4685 | 0.2621 | 97.4542 | 0.1656 | 98.5981 | 0.1019 | 99.2371 | 0.0615 |
| 31 | 95.2064 | 0.2726 | 97.2886 | 0.1739 | 98.4962 | 0.1080 | 99.1756 | 0.0658 |
| 32 | 94.9338 | 0.2835 | 97.1147 | 0.1826 | 98.3882 | 0.1145 | 99.1098 | 0.0704 |
| 33 | 94.6503 | 0.2948 | 96.9321 | 0.1917 | 98.2737 | 0.1213 | 99.0394 | 0.0753 |
| 34 | 94.3555 | 0.3066 | 96.7404 | 0.2013 | 98.1524 | 0.1286 | 98.9641 | 0.0806 |
| 35 | 94.0489 | 0.3188 | 96.5391 | 0.2113 | 98.0238 | 0.1363 | 98.8835 | 0.0862 |
| 36 | 93.7301 | 0.3316 | 96.3278 | 0.2219 | 97.8875 | 0.1445 | 98.7973 | 0.0923 |

100-YEAR LIFE.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 37 | 93.3985 | 0.3449 | 96.1059 | 0.2330 | 97.7430 | 0.1532 | 98.7050 | 0.0987 |
| 38 | 93.0536 | 0.3587 | 95.8729 | 0.2447 | 97.5898 | 0.1624 | 98.6063 | 0.1056 |
| 39 | 92.6949 | 0.3730 | 95.6282 | 0.2569 | 97.4274 | 0.1721 | 98.5007 | 0.1130 |
| 40 | 92.3219 | 0.3879 | 95.3713 | 0.2697 | 97.2553 | 0.1824 | 98.3877 | 0.1209 |
| 41 | 91.9340 | 0.4034 | 95.1016 | 0.2832 | 97.0729 | 0.1934 | 98.2668 | 0.1294 |
| 42 | 91.5306 | 0.4196 | 94.8184 | 0.2974 | 96.8795 | 0.2050 | 98.1374 | 0.1385 |
| 43 | 91.1110 | 0.4364 | 94.5210 | 0.3123 | 96.6745 | 0.2173 | 97.9989 | 0.1482 |
| 44 | 90.6746 | 0.4538 | 94.2087 | 0.3279 | 96.4572 | 0.2303 | 97.8507 | 0.1585 |
| 45 | 90.2208 | 0.4720 | 93.8808 | 0.3443 | 96.2269 | 0.2441 | 97.6922 | 0.1696 |
| 46 | 89.7488 | 0.4908 | 93.5365 | 0.3615 | 95.9828 | 0.2588 | 97.5226 | 0.1815 |
| 47 | 89.2580 | 0.5105 | 93.1750 | 0.3795 | 95.7240 | 0.2743 | 97.3411 | 0.1942 |
| 48 | 88.7475 | 0.5309 | 92.7955 | 0.3985 | 95.4497 | 0.2908 | 97.1469 | 0.2078 |
| 49 | 88.2166 | 0.5521 | 92.3970 | 0.4184 | 95.1589 | 0.3082 | 96.9391 | 0.2223 |
| 50 | 87.6645 | 0.5742 | 91.9786 | 0.4394 | 94.8507 | 0.3267 | 96.7168 | 0.2379 |
| 51 | 87.0903 | 0.5972 | 91.5392 | 0.4613 | 94.5240 | 0.3463 | 96.4789 | 0.2546 |
| 52 | 86.4931 | 0.6211 | 91.0779 | 0.4844 | 94.1777 | 0.3671 | 96.2243 | 0.2724 |
| 53 | 85.8720 | 0.6459 | 90.5935 | 0.5086 | 93.8106 | 0.3891 | 95.9519 | 0.2915 |
| 54 | 85.2261 | 0.6718 | 90.0849 | 0.5340 | 93.4215 | 0.4124 | 95.6604 | 0.3119 |
| 55 | 84.5543 | 0.6986 | 89.5509 | 0.5607 | 93.0091 | 0.4372 | 95.3485 | 0.3337 |
| 56 | 83.8557 | 0.7266 | 88.9902 | 0.5888 | 92.5719 | 0.4634 | 95.0148 | 0.3570 |
| 57 | 83.1291 | 0.7556 | 88.4014 | 0.6182 | 92.1085 | 0.4912 | 94.6578 | 0.3820 |
| 58 | 82.3735 | 0.7859 | 87.7832 | 0.6491 | 91.6173 | 0.5207 | 94.2758 | 0.4088 |
| 59 | 81.5876 | 0.8173 | 87.1341 | 0.6816 | 91.0966 | 0.5519 | 93.8670 | 0.4374 |
| 60 | 80.7703 | 0.8500 | 86.4525 | 0.7157 | 90.5447 | 0.5851 | 93.4296 | 0.4680 |
| 61 | 79.9203 | 0.8840 | 85.7368 | 0.7515 | 89.9596 | 0.6202 | 92.9616 | 0.5008 |
| 62 | 79.0363 | 0.9193 | 84.9853 | 0.7890 | 89.3394 | 0.6574 | 92.4608 | 0.5358 |
| 63 | 78.1170 | 0.9561 | 84.1963 | 0.8285 | 88.6820 | 0.6968 | 91.9250 | 0.5733 |
| 64 | 77.1609 | 0.9944 | 83.3678 | 0.8699 | 87.9852 | 0.7386 | 91.3517 | 0.6135 |
| 65 | 76.1665 | 1.0341 | 82.4979 | 0.9134 | 87.2466 | 0.7829 | 90.7382 | 0.6564 |
| 66 | 75.1324 | 1.0755 | 81.5845 | 0.9591 | 86.4637 | 0.8299 | 90.0818 | 0.7024 |
| 67 | 74.0579 | 1.1185 | 80.6254 | 1.0070 | 85.6338 | 0.8797 | 89.3794 | 0.7515 |
| 68 | 72.9384 | 1.1633 | 79.6184 | 1.0574 | 84.7541 | 0.9325 | 88.6279 | 0.8041 |
| 69 | 71.7751 | 1.2098 | 78.5610 | 1.1102 | 83.8216 | 0.9884 | 87.8238 | 0.8604 |
| 70 | 70.5653 | 1.2582 | 77.4508 | 1.1657 | 82.8332 | 1.0477 | 86.9634 | 0.9206 |

100-Year Life.—(*Continued.*)

| Age, in years. | Interest Rate. 4% | | Interest Rate. 5% | | Interest Rate. 6% | | Interest Rate. 7% | |
|---|---|---|---|---|---|---|---|---|
| | Value. | Dep. | Value. | Dep. | Value. | Dep. | Value. | Dep. |
| 71 | 69.3071 | 1.3085 | 75.2851 | 1.2240 | 81.7855 | 1.1106 | 86.0428 | 0.9851 |
| 72 | 67.9986 | 1.3608 | 75.0611 | 1.2852 | 80.6749 | 1.1772 | 85.0577 | 1.0540 |
| 73 | 66.6378 | 1.4153 | 73.7759 | 1.3495 | 79.4977 | 1.2479 | 84.0037 | 1.1278 |
| 74 | 65.2225 | 1.4719 | 72.4264 | 1.4170 | 78.2498 | 1.3228 | 82.8759 | 1.2068 |
| 75 | 63.7506 | 1.5308 | 71.0094 | 1.4878 | 76.9270 | 1.4022 | 81.6691 | 1.2912 |
| 76 | 62.2198 | 1.5920 | 69.5216 | 1.5622 | 75.5248 | 1.4863 | 80.3779 | 1.3816 |
| 77 | 60.6278 | 1.6557 | 67.9594 | 1.6403 | 74.0385 | 1.5754 | 78.9963 | 1.4783 |
| 78 | 58.9721 | 1.7219 | 66.3191 | 1.7223 | 72.4631 | 1.6700 | 77.5180 | 1.5818 |
| 79 | 57.2502 | 1.7908 | 64.5968 | 1.8084 | 70.7931 | 1.7701 | 75.9362 | 1.6925 |
| 80 | 55.4594 | 1.8624 | 62.7884 | 1.8989 | 69.0230 | 1.8764 | 74.2437 | 1.8110 |
| 81 | 53.5970 | 1.9369 | 60.8895 | 1.9938 | 67.1466 | 1.9889 | 72.4327 | 1.9378 |
| 82 | 51.6601 | 2.0144 | 58.8957 | 2.0935 | 65.1577 | 2.1083 | 70.4949 | 2.0734 |
| 83 | 49.6457 | 2.0950 | 56.8022 | 2.1982 | 63.0494 | 2.2348 | 68.4215 | 2.2186 |
| 84 | 47.5507 | 2.1788 | 54.6040 | 2.3081 | 60.8146 | 2.3689 | 66.2029 | 2.3739 |
| 85 | 45.3719 | 2.2659 | 52.2959 | 2.4235 | 58.4457 | 2.5110 | 63.8290 | 2.5400 |
| 86 | 43.1060 | 2.3566 | 49.8724 | 2.5447 | 55.9347 | 2.6616 | 60.2890 | 2.7179 |
| 87 | 40.7494 | 2.4508 | 47.3277 | 2.6719 | 53.2731 | 2.8213 | 58.5711 | 2.9081 |
| 88 | 38.2986 | 2.5489 | 44.6558 | 2.8055 | 50.4518 | 2.9906 | 55.6630 | 3.1117 |
| 89 | 35.7497 | 2.6508 | 41.8503 | 2.9458 | 47.4612 | 3.1701 | 52.5513 | 3.3295 |
| 90 | 33.0989 | 2.7568 | 38.9045 | 3.0931 | 44.2911 | 3.3603 | 49.2218 | 3.5625 |
| 91 | 30.3421 | 2.8671 | 35.8114 | 3.2477 | 40.9308 | 3.5619 | 45.6593 | 3.8119 |
| 92 | 27.4750 | 2.9818 | 32.5637 | 3.4101 | 37.3689 | 3.7756 | 41.8474 | 4.0788 |
| 93 | 24.4932 | 3.1011 | 29.1536 | 3.5806 | 33.5933 | 4.0021 | 37.7686 | 4.3643 |
| 94 | 21.3921 | 3.2251 | 25.5730 | 3.7597 | 29.5912 | 4.2423 | 33.4043 | 4.6698 |
| 95 | 18.1670 | 3.3541 | 21.8133 | 3.9477 | 25.3489 | 4.4968 | 28.7345 | 4.9967 |
| 96 | 14.8129 | 3.4883 | 17.8656 | 4.1450 | 20.8521 | 4.7666 | 23.7378 | 5.3464 |
| 97 | 11.324[illegible] | 3.6278 | 13.7206 | 4.3523 | 16.0855 | 5.0526 | 18.3914 | 5.7207 |
| 98 | 7.6968 | 3.7729 | 9.3683 | 4.5699 | 11.0329 | 5.3558 | 12.6707 | 6.1211 |
| 99 | 3.9239 | 3.9239 | 4.7984 | 4.7984 | 5.6771 | 5.6771 | 6.5496 | 6.5496 |
| 100 | 0.0000 | | 0.0000 | | 0.0000 | | 0.0000 | |
| | | 100.0000 | | 100.0000 | | 100.0000 | | 100.0000 |

# APPENDIX II.

## Some Examples of the Expectation of Life of So-Called Permanent Structures.

When one examines records of the life of different kinds of so-called permanent structures, the actual life of such structures is frequently found to be much shorter than the life usually estimated, and probably much shorter than was anticipated when the works were new. For the purpose of illustration, this subject has been investigated along two lines: (*a*) the actual life of railway stations, and especially of terminal stations; (*b*) the life of water-works sources, reservoirs, and pumping stations.

TABLE 5.—Life of Railway Stations.

| Location. | Year when built or enlarged. | Life, in years. | Remarks. |
|---|---|---|---|
| **Grand Central Station and Predecessors, New York City.** | | | |
| 241 Bowery, at Prince Street | 1832 | | |
| Tryon Row (Municipal Building) | 1839 | 7 | |
| Fourth Avenue and Twenty-sixth Street (Madison Square Garden) | 1857 | 18 | |
| Forty-second Street | 1873 | 16 | |
| Forty-second Street. Extensive enlargement | 1884 | | |
| Forty-second Street Station. Entirely remodeled and three stories added for offices | 1899 | 34 | |
| Construction of new terminal started | 1903 | | |
| Present station first used for electric service | 1907 | | |
| Present station in full use | 1912 | | |
| **Boston and Albany Railroad, Albany, N. Y.** | | | |
| Wooden building at Colonie and Broadway | 1866 | | |
| Brick and stone building at Maiden Lane | 1873 | 7 | |
| Existing stone station | 1900 | 27 | |
| **Boston and Albany Railroad, Boston, Mass.** | | | |
| Original station | 1834 | ... | History between these dates not known. |
| Kneeland Street Station | 1881 | | |
| South Station (Union Station) | 1899 | 18 | |
| **Boston and Providence Railroad, Boston, Mass.** | | | |
| Original station | 1834 | | History not known. |
| Park Square Station | 1874 | | |
| South Station | 1899 | 25 | |

TABLE 5.—LIFE OF RAILWAY STATIONS.—(*Continued.*)

| Location. | Year when built or enlarged. | Life, in years. | Remarks. |
|---|---|---|---|
| **NEW YORK AND NEW ENGLAND RAILROAD, BOSTON, MASS.** | | | |
| Original station | 1855 | 17 | |
| Station burned and rebuilt | 1872 | | |
| Station extensively remodeled | 1880 | 27 | |
| South Station | 1899 | | |
| **BOSTON AND MAINE RAILROAD, BOSTON, MASS.** | | | |
| Haymarket Square Station | 1846 | 47 | |
| North Station (Union Station) | 1893 | | |
| **EASTERN RAILROAD, BOSTON, MASS.** | | | |
| Causeway Street Station | 1854 | 9 | |
| Station burned and replaced by brick building | 1863 | 30 | |
| North Station | 1893 | | |
| **FITCHBURG RAILROAD, BOSTON, MASS.** | | | |
| Causeway Street Station | 1848 | 45 | |
| North Station | 1893 | | |
| **BOSTON AND LOWELL RAILROAD, BOSTON, MASS.** | | | |
| Minot Street Station | 1835 | 17 | |
| Causeway Street Station | 1852 | | |
| Causeway Street Station rebuilt | 1873 | 21 | |
| Merged into North Station, with minor alterations | 1893 | | |
| **BOSTON AND ALBANY RAILROAD, WORCESTER, MASS.** | | | |
| Original station | 1835 | | History not known. |
| Union Station | 1875 | 36 | |
| New Union Station | 1911 | | |
| Average life of all stations | | 24 | |

The Forty-second Street Station, in New York City, although credited with a total life of 34 years, had that length of life only for the part built in 1883. The extensive enlargement in 1884 had a life of 23 years and the remodeled station of 1899, had a life of but 8 years.

### Life of Water-Works Structures.

The average life of the sources of water supply given in Table 6 was shortened by the introduction of a general system of water supply for the Metropolitan District, which made the further use of small local supplies undesirable.

Lake Cochituate, the original source of supply of Boston, has been in use 65 years, and the Sudbury River, the second source of supply of the city, has been in use 36 years, but both these sources are now used only to a limited extent to supplement, when necessary, the supply from the Wachusett Reservoir, the newest source.

TABLE 6.—Sources of Supply.

Massachusetts Metropolitan Water District.

| Source. | Period. Years. | Life, in years. | Remarks. |
|---|---|---|---|
| Boston, Mystic Lake source, supplying 12 000 000 gal. daily in the last year of its life | 1864-1898 | 34 | Abandoned on account of increasing pollution. |
| Malden, Spot Pond | 1870-1898 | 28 | |
| Malden, ground-water source | 1890-1900 | 10 | |
| Quincy, ground-water source | 1884-1898 | 14 | |
| Quincy, storage reservoir | 1888-1898 | 10 | |
| Hyde Park, ground-water source, near Neponset River | 1885-1912 | 27 | |
| Hyde Park, ground-water source, near Mother Brook | 1899-1912 | 13 | |
| Medford, Spot Pond | 1870-1898 | 28 | |
| Medford, storage reservoir | 1894-1899 | 5 | |
| Revere, ground-water source in town | 1884-1898 | 14 | |
| Revere, ground-water source in Cliftondale | 1891-1899 | 8 | |
| Melrose, Spot Pond | 1870-1898 | 28 | |
| Watertown, ground-water source | 1885-1898 | 13 | |
| Arlington, storage reservoir | 1872-1899 | 27 | |
| Arlington, ground-water source | 1895-1900 | 5 | |
| Swampscott, ground-water source | 1885-1899 | 14 | |
| Lexington, ground-water source | 1884-1903 | 19 | |
| Lexington, storage reservoir | 1894-1903 | 9 | |
| Average | ........... | 17 | |

### Reservoirs.

At the time of the completion of the original works supplying Boston, in 1848, there was a reservoir of 23 acres in Brookline at the end of the brick aqueduct. The pipes from this reservoir led to three distributing reservoirs: one of them, on Beacon Hill in Boston, was a large elevated masonry reservoir supported by arches; the other two were earthen reservoirs on hills in South Boston and East Boston.

The Brookline Reservoir lost a large part of its value at the end of 22 years, when it was superseded by a larger reservoir, but it had a life of 52 years before its use was discontinued. The use of Beacon Hill Reservoir was discontinued in 23 years; of the South Boston

Reservoir in 24 years; and of the East Boston Reservoir, except for the purpose of emergency use, in 32 years.

As in the case of the sources of supply, the life of pumping stations in the Massachusetts Metropolitan Water District, has, in many cases. been shortened by the introduction of a general system of supply. The oldest of the five existing pumping stations in this district was built 26 years ago, and the building was extended 11 years after it was built.

A study of other water systems shows that sources, pumping stations, reservoirs and other works are frequently abandoned for various reasons, such as the pollution of sources, the growth of population at higher elevations than those originally provided for, and the general improvement and enlargement of the water system to meet changing conditions.

An instance of an extremely short life of a reservoir and pumping station was observed in connection with a valuation for rate-making made in 1909. The rates in question were those of 1904 and 1905. The pumping station and reservoir did not appear on the inventory for 1904, because they had not then been built. They did appear on the inventory for 1905, but could not be seen by the appraiser in 1909 because they had been superseded and removed. The cause of the short life was a great influx of population which settled on elevated land and required a more complete system of high-service works.

TABLE 7.—Pumping Stations.

| | Period. Years. | Life, in years. |
|---|---|---|
| Boston, original high service | 1870-1888 | 18 |
| Old East Boston | 1880-1889 | 9 |
| New East Boston | 1889-1898 | 9 |
| West Roxbury | 1886-1913 | 27 |
| Mystic { Worthington Pump | 1864-1898 | 34 |
| Mystic { Leavitt Pump* | 1896-1898 | 2 |
| Somerville | 1890-1900 | 10 |
| Malden, Spot Pond Station | 1883-1898 | 15 |
| Malden, Webster Park Station | 1800-1900 | 10 |
| Chelsea | 1886-1900 | 14 |
| Everett | 1888-1899 | 11 |
| Quincy | 1884-1898 | 14 |
| Hyde Park, Neponset River Station | 1885-1912 | 27 |
| Hyde Park, Mother Brook Station | 1899-1912 | 13 |
| Medford, Spot Pond Station | 1892-1898 | 6 |
| Medford, Reservoir Station | 1894-1899 | 5 |
| Revere, Town Station | 1884-1898 | 14 |
| Revere, Cliftondale Station | 1891-1899 | 8 |
| Melrose | 1886-1899 | 13 |
| Watertown | 1885-1898 | 13 |
| Arlington | 1895-1900 | 5 |
| Swampscott | 1885-1899 | 14 |
| Lexington | 1884-1903 | 19 |
| Average | ............ | 14 |

*This pumping engine was transferred to another pumping station when the Mystic Station was abandoned.

On the other hand it is not to be inferred from Table 6, that the average life of sources of water supply is but 17 years, or from Table 7, that the average life of pumping stations is but 14 years—for the life history of the majority of water-works will show that such is not the case. These examples have been cited merely to indicate that functional depreciation is an active force to be considered and given such practical weight as circumstances and experience may indicate to be fair.

Thus, by comparison with Table 7, there might be cited the experience of the Spring Valley Water Company, of San Francisco, Cal. Out of eight pumping stations (excluding the centrifugal booster and ground-water supply stations, recently erected and still in service), all but one are in active service—and this one was an emergency station, and not one of permanent construction. The age of the oldest station is about 30 years, the mean age of the investment in the eight stations, 15 years.

Table 8 is a statement of the pumps in these stations.

TABLE 8.

| Fly-wheel pumps. | | Direct-acting pumps. | |
|---|---|---|---|
| Black Point | 1-30 years. | Ocean View | 2-18 years. |
| " " | 2-22 " | Precita | 1-17 " |
| Belmont | 2-28 " | " | 1- 8 " |
| " | 3-15 " | Centrifugal pumps. | |
| Merced | 2-25 " | Ravenswood | 1- 4 years. |
| Millbrae | 1-18 " | Pleasanton | 1- 3 " |
| Central | 1-18 " | " | 1- 7 " |
| Clarendon | 1-15 " | Crystal Springs | 3- 3 " |

No pumps have been discarded, except three small double-acting low-duty pumps at Bald Hill emergency station, used in two or three dry years for a few months only.

This comparison is made merely to indicate that each problem must be thoroughly studied in the light of the local past, present, and probable future conditions.

## APPENDIX III.

### SOME EXAMPLES OF ACTUAL OVERHEAD COST.

In presenting these data as to actual overhead cost, the Committee desires to lay stress on the need of using all such figures with caution. Items entering into overhead cost of different and similar works are rarely alike, and many of the expenses, and even classes of expense, incurred on one piece of work, might not be incurred on another. It is impossible to forecast all the conditions and items of overhead cost which will develop on any work. Were it otherwise, no separate group of overhead cost would be necessary, for every expense would be included in the unit costs.

Experience has shown, however, that indeterminate or indeterminable expenses always occur, and in such amount that they frequently aggregate like, or similar, percentages of fixed unit costs. Hence, the mere amount of those percentages, developed on other works, has a bearing on the problem. Their segregation aids in forecasting probable developments the more accurately, and they furnish a background of comparative experience which is useful.

Moreover, in valuing properties, years after their construction, grave uncertainties generally arise as to the conditions which actually developed during the construction of these works, and records are virtually never in sufficient detail to eliminate them. Therefore, the evaluator, as the designing and constructing engineer, must make reasonable allowance, to cover the effect of these uncertainties, if justice is to be done.

To the evaluator, the records of greatest value are those developed on work, with all the details, conditions, and surroundings of which he is personally familiar, as these he is in a position to use most intelligently as a yard-stick, and is least likely to misapply. The evaluator should not attempt to figure the value of property with the construction costs of which he is not familiar.

With reference to the data submitted, the details presented to the Committee were so voluminous as to be impracticable of publication.

(*a*).—EXAMPLES SHOWING CUMULATIVE ENGINEERING COSTS, WITH VARYING VOLUMES OF WORK.

Compiled from printed reports, under the supervision of Alfred Craven, M. Am. Soc. C. E.

| Year. | New York State Barge Canal. | Board of Water Supply—New York Aqueduct. |
|---|---|---|
| 1905 | 750% | ..... |
| 1906 | 98 | 167.0% |
| 1907 | 40 | 174.0 |
| 1908 | 25 | 81.3 |
| 1909 | 14 | 32.9 |
| 1910 | 13 | 19.2 |
| 1911 | 9 | 14.5 |
| 1912 | 9 | 12.5 |
| 1913 | 9 | 11.86 |

(*b*).—Examples of Overhead Costs on Water-Works Construction.

Administration, Engineering, Miscellaneous, Etc., Excluding Taxes, Interest-during-Construction, and Discount and Expenses on Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Year. | Locality and character of works. | Total construction cost excl. overhead and interest and land. | Administration. | Engineering and superintendence. | Police supervision. | Miscellaneous. | Total excluding taxes, contingencies, and interest. | Authority and source of information, and remarks. |
|---|---|---|---|---|---|---|---|---|
| 1914 | New York Catskill Supply, on structures only: | | | | | | | |
| ...... | For engineering base... | $108 144 219 | 1.26% | 11.12% | 2.18% | Excluded. | *14.56% | *Excludes 0.5% Preliminary Engineering Report. Based on Annual Report of Board of Water Supply. Interest during construction, 18.02%, giving **total, 35.2%** (Hazen). |
| | " police base......... | $99 393 000 | | | | | | |
| | " administration...... | $113 861 751 | | | | | | |
| | On land, including overhead and interest...... | $21 196 700 | ........ | ........ | ........ | ........ | 43.08% | Including interest and taxes to January 1st, 1916, when works go into service, 37.15%, the total overhead and interest = 96.23% (Hazen's analysis). |
| 1895 | Metropolitan Water-Works (Boston)................ | $23 600 000 | ........ | ........ | ........ | ........ | 16.65% | |
| | | 24 659 000 | ........ | 8.42% | ........ | ........ | ........ | F. P. Stearns, M. Am. Soc. C. E., Val. Com. Rep. p. 23. Metcalf finds Int-dur-const. 16.8% at 3.5% annual rate basis. If it is to be assumed that the cast-iron distribution service went into immediate use 6 months after construction, this would be 11.4%, and if premiums be accounted 10.8% approx., excluding administration costs from the base. The total overhead and interest is then 16.5 + 16.8% = **33.3%** (or 27.9%). |
| 1901–04 | Weston Aqueduct.......... | 2 334 654 | ........ | 11.4% | Legal | 0.8% | ........ | |
| 1895–98 | Wachusett Dam............ | 2 118 496 | ........ | 11.4% | | | ........ | |
| | Wachusett Aqueduct....... | 1 540 388 | ........ | 10.6% | Legal | 0.8% | ........ | |
| 1899–1900 | Main distribution pipes.... | 2 117 173 | ........ | 7.8% | ........ | ........ | ........ | |
| 1900 | Sudbury Dam and Reservoir.............. | 2 125 526 | ........ | 8.1% | ........ | ........ | ........ | |
| 1902 | Chestnut Hill Pumping Station.............. | 419 013 | ........ | 9.5% | ........ | ........ | ........ | |
| 1897–Dec. 31, 1907 | Cincinnati Water-Works, Ohio.............. | $9 469 918 | ........ | | | ........ | 12.72% | Hazen's analysis. Interest during construction 15.24%, making total overhead and interest **29.90%**. |
| | do. excluding real estate, overhead, and interest.... | $9 279 203 | 4.5% | 9.1% | Legal | 0.4% | 14.0% | Metcalf incl. 0.9% prelim. engr. and experimental plant and 0.4% legal. Grand total cost excl. interest $10 844 331. |
| 1910 | Pittsfield, Mass., Water-Works.............. | 463 751 | ........ | 13.8% | ........ | ........ | ........ | Metcalf. |
| 1906 | Kennebec Water District, Waterville, Me......... | 267 408 | 2.53% | 7.35% | ........ | ........ | ........ | Metcalf. |
| 1910 | Springfield, Mass., Water-Works: | | | | | | | |
| | (*a*) New Little River Supply, dam, filters, conduits............ | 2 000 000 | ........ | ........ | ........ | ........ | 14.30% | (Hazen) on new supply. |
| 1906 | (*b*) Ludlow filters........ | 43 306 | ........ | 17.0% | ........ | ........ | ........ | Emergency supply. |
| 1911 | Des Moines, Ia., Water-Works, new source........ | ........ | ........ | 8.6% | ........ | ........ | ........ | Hazen. |

(c).—EXAMPLES OF OVERHEAD COSTS ON VARIOUS WORKS.—(*Continued*).

Administration, Engineering, Miscellaneous, Etc., Excluding Taxes, Interest-during-Construction, and Discount and Expenses upon Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Year. | Locality and character of works. | Total construction cost excl. overhead and int. and taxes. | Administration. | Engineering and superintendence. | General miscellaneous and incidental expense. | Total, excluding taxes, contingencies, and interest. | Authority, source of information, and remarks. |
|---|---|---|---|---|---|---|---|
| 1867-1901. | St. Louis, Mo., Water-Works... | $8 472 576 | ...... | 11.2% | ...... | ...... | Report on water supply of City of St. Louis, 1902, App. D., p. 139. |
| 1905-1914. | Los Angeles, Cal., Aqueduct... | 22 540 580 | ...... | 8.8% | ...... | ...... | NOTE.—Of the total cost exclusive of interest ($24 526 868), 22.3% covered the auxiliary or incidental construction. |
| **SLOW SAND FILTER PLANTS.** | | | | | | | |
| 1903...... | Watertown, N. Y., filters......... | $97 065 | ...... | 5.9% | ...... | ...... | Allen Hazen. |
| 1910...... | Ogdensburg, N. Y., filters......... | 167 694 | ...... | 7.0% | ...... | ...... | " " |
| 1904...... | Hudson River State Hospital filters.................. | 36 000 | ...... | 10.5% | ...... | ...... | " " |
| 1908...... | Peekskill, N. Y., filters........... | 63 306 | ...... | 7.1% | ...... | ...... | " " |
| 1903...... | Yonkers, N. Y., open filters...... | 50 165 | ...... | 7.3% | ...... | ...... | " " |
| 1907...... | Yonkers, N. Y., covered filters.. | 106 708 | ...... | 8.7% | ...... | ...... | " " |
| 1903...... | Ithaca, N. Y..................... | 192 114 | ...... | 7.0% | ...... | ...... | " " |
| 1903-05... | Washington, D. C., filters....... | 3 356 000 | ...... | 7.4% | ...... | ...... | " " |
| 1909-12... | Toronto, Canada, filters......... | 781 519 | ...... | 10.2% | ...... | ...... | (Excluding work done City Engineer's Corps.) |
| 1897-99... | Albany, N. Y., filters............ | 460 600 | ...... | 6.7% | ...... | ...... | *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 294. |
| 1896...... | Ashland, Wis., filters............ | 38 378 | ...... | 4.7% | ...... | ...... | Excluding work of Company employees, and administration and miscellaneous. (William Wheeler.) |
| 1904-08. | Columbus, Ohio, improved water-works...................... | 1 189 210 | ...... | 8.1% | ...... | ...... | *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 261. |

(c).—EXAMPLES OF OVERHEAD COSTS ON VARIOUS WORKS.

Administration, Engineering, Miscellaneous, Etc., Excluding Taxes, Interest-during-Construction, and Discount and Expenses on Bonds and other Securities Issued.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Year. | Locality and character of works. | Total construction cost, excl. overhead and int. and taxes. | Administration. | Engineering and superintendence. | General miscellaneous and incidental expense. | Total, excluding taxes, contingencies, and interest. | Authority, source of information, and remarks. |
|---|---|---|---|---|---|---|---|
| 1909* | Charles River Basin Commission, Boston. Dam, locks, embankments, conduits......... | $3 215 656 | 3.2% | 15.5% | ...... | 18.7% | |
| 1906-1912 | Louisville, Ky. Sewers......... | $3 289 331 | 2.37% | { 1.70% Prel. 10.23% | 1.09% | 15.39% | Harrison P. Eddy. |
| 1895-1912 | Boston Transit Commission, subways...................... | $3 586 002 | ...... | 11.34% | 3.68% | 15.02% | Am. Soc. C. E. Valuation Com. Report, 1913, p. 24. |
| | East Boston tunnels............ | 2 894 595 | ...... | 6.62% | 5.57% | 12.19% | |
| | Boston tunnel and subway.... | 7 623 207 | ...... | 5.48% | 3.58% | 9.06% | |
| | Cambridge connection.......... | 1 199 904 | ...... | 8.05% | 5.20% | 13.25% | |
| | | $15 303 708 | Average | 7.28% | 3.80% | 11.09% | |
| | Pennsylvania Railroad tunnels, East River.................... | .......... | ...... | 6.1% | ...... | ...... | |
| 1885 | Washington Bridge over Harlem River, New York City.... | $2 648 785 | 1.5% | 6.1% | ...... | ...... | Description by Hutton. |
| | Pacific Gas and Electric Company. Rate suit data (San Francisco, Cal.) | | | | | | |
| | (a) Big Creek.............. | $11 339 308 | 4.76% | 6.23% | 3.17% | 14.16%* | Int. 8.86% { making combined overhead and interest exclusive of taxes and contingencies..................23.02% |
| | (b) Big Bend, permanent dams................... | 522 437 | 8.08% | 9.30% | 1.70% | 19.08%* | Int. 3.68% do. 22.76% |
| | (c) Big Bend extension..... | 476 928 | 5.23% | 6.40% | 1.83% | 13.46%* | " 7.10% do. 20.56% |
| | (d) Kaweah No. 3.......... | 511 560 | 2.88% | 7.06% | 4.35% | 14.29%* | " 7.38% do. 21.67% |
| | (e) Kern Canyon............ | 340 685 | 1.11% | 12.33% | 0.14% | 13.68%* | " 7.21% do. 20.79% |
| | (f) San Joaquin............ | 1 563 485 | 1.00% | 9.5% | 2.0% | 12.50%* | " 6.0% do. 18.50% |
| | (g) Tule River.............. | 649 096 | 2.89% | 11.63% | 1.76% | 16.28%* | " 9.96% do. 26.24% |
| | (h) Spaulding drum development.............. | 3 498 894 | 4.30% | 6.0% | 1.40% | 11.70%* | " 6.8% do. 18.50% |

* G. P. Cutten, Esq., Counsel to the Company, who furnished these records, writes (August 14th, 1915): "It must be borne in mind that the construction costs in this table are actual costs taken from the books, and all contingencies, etc., which occurred on the jobs are in the unit costs. An evaluator must make separate allowance for these contingencies in his overhead, which would increase the overhead on a comparative basis from 5 to 10 per cent.

(*d*).—Overhead and Interest During Construction on Four Large Undertakings.

Summary of Analysis made by Allen Hazen, M. Am. Soc. C. E.

| No. | City and works. | Amount. | Period. | Land only. | Structures only. |
|---|---|---|---|---|---|
| 1 | Board of Water Supply, New York City. | $135 000 000 | 1905-1915 | ......... | ......... |
| | Engineering and administration..... | ......... | ......... | 43.08% | 14.56% |
| | Interest at 4.30%, and taxes during construction ......... | ......... | ......... | 37.15% | 18.02% |
| | Total overhead......... | ......... | ......... | 96.23% | 35.20% |
| 2 | Metropolitan Water Board, Boston...... | $23 600 000 | 1896-1906 | ......... | ......... |
| | Engineering and administration..... | ......... | ......... | ......... | 16.65% |
| | Interest at 3.10% during construction......... | | | ......... | 9.42%* |
| | Total overhead......... | ......... | ......... | ......... | 27.64% |
| 3 | Cincinnati Water-Works Improvements. | $11 000 000 | 1897-1908 | ......... | ......... |
| | Engineering and administration..... | ......... | ......... | ......... | 12.72%* |
| | Interest at 3.75% during construction......... | ......... | ......... | ......... | 15.24% |
| | Total overhead......... | ......... | ......... | ......... | 29.90% |
| 4 | Little River Works, Springfield, Mass... | $2 000 000 | 1907-1908 | ......... | ......... |
| | Engineering and administration..... | ......... | ......... | ......... | 14.30% |
| | Interest at 3.5% during construction, about......... | ......... | ......... | ......... | 3.50% |
| | Total overhead, about......... | ......... | ......... | ......... | 18.3% |

* Calculated on structures and land taken together.

Overhead on Structures with Engineering and Administration as it Actually was, with Interest Calculated at Actual Rate and at a 6% Rate.

| No. | | Figures in percentages. | | | | |
|---|---|---|---|---|---|---|
| | | New York Board of Water Supply, structures. | Boston Metropolitan Water Board, structures and land. | Cincinnati Water-Works, structures and land. | Little River Water-Works, Springfield, Mass., structures only. | New York Board of Water Supply, land. |
| 1 | Engineering and administration...... | 14.56% | 16.65% | 12.72% | 14.30% | 43.08% |
| 2 | Interest rate......... | 4.30% | 3.10% | 3.75% | 3.50% | 4.30% |
| 3 | Total actual interest......... | 18.02% | 9.42% | 15.24% | 3.50% | 37.15%* |
| 4 | Average number of years for which interest was paid......... | 4.19% | 3.04% | 4.07% | 1.00% | 7.85% |
| 5 | Interest for the same period at 6%, obtained by multiplying actual interest by ratio of 6% to rate paid... | 25.15% | 18.23% | 24.39% | 6.00% | 47.00% |
| 6 | Corresponding total overhead......... | 43.87% | 37.91% | 40.20% | 21.16% | 110.3% |

* Includes taxes.

## (e).—Interest During Construction on Various Works.

### Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

| No. | Locality and character of works. | Period of years. | Date of completion. December 31st. | Reported total cost. | Accrued Interest during Construction to Completion of Structure with Compound Interest at: Assumed actual rate of: | Actual rate. | 5% | 6% | Remarks. |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Board of Water Supply, New York City. Reservoirs, dams, aqueducts, etc. | | | | | | | | |
| | (a) Water-works structures only | 12 | 1915 | $132 760 260 | 4.30% | 18.02% | 25.15% | ........ | Annual Reports. |
| | (b) On land | 10 | 1915 | 21 196 700 | 4.30 | 37.15 | 47.0 | ........ | Allen Hazen, M. Am. Soc. C. E. |
| | | | | | 4.30 / 6 | 31.4 | ........ | ........ | Excluding in rate the interest upon award. / Including do. |
| 2 | Contract No. 80. Grade and pressure tunnel | 4 | 1913 | 420 465 | 4 ± ? | 9.0 | 11.3 | 13.7% | |
| 3 | Contract No. 45. Cut and cover aqueduct | 5 | 1913 | 1 592 054 | 4 ± ? | 8.8 | 11.1 | 13.5 | |
| 4 | Contract No. 12. Rondout Siphon and tunnel | 6 | 1913 | 6 515 444 | 4 ± ? | 14.7 | 18.7 | 22.8 | |
| 5 | Contract No. 20. (4.8 miles.) Wooden pressure tunnel | 5 | 1913 | 3 344 497 | 4 ± ? | 10.5 | 13.3 | 16.2 | Excluding land, engineering, administration, and interest. |
| 6 | Ashokan Main Dams | 9 | 1914 | 11 773 216 | .......... | 16.5 | 20.5 | 25.7 | do. |
| 7 | Massachusetts Metropolitan Water-Works. (Boston.) | See later estimate on entire plant. | | | | | | | |
| 8 | Wachusett Aqueduct | 3 | 1898 | 1 713 405* | .......... | 5.0 | 8.0 | 10.0 | *Excluding interest. |
| 9 | Wachusett Dam | 4 | 1898 | 1 713 405 | .......... | 5.0 | 8.0 | 10.0 | Note.—Further construction, up to total $2 200 000, was executed later. |
| 10 | Weston Aqueduct | ........ | ........ | 2 783 523* | .......... | 7.3 | 11.3 | 13.6 | |
| 11 | Basin 5, earth reservoir | 5 | 1894 | 910 000 | 4 ± ? | 12.3 | 15.6 | 19.0 | |
| 12 | Massachusetts Metropolitan Sewerage Contract No. 66: 5 300 ft. rock tunnel, 9 ft. 3 in. by 10 ft. 2 in. | 4 | 1903 | 252 861 | .......... | 3.7 | 6.1 | 7.3 | |
| 13 | Boston Transit Commission. East Boston Tunnel | 7 | 1904 | 2 571 185 | 3½ ± | 9.8 | 13.5 | 16.4 | |

## (e).—Interest During Construction.—(*Continued*).

Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

| No. | Locality and character of works (structures only). | Period of years. | Date of completion, December 31st. | Reported total cost, when put into service, excluding interest during construction. | Accrued Interest During Construction to Completion of Structure, with Interest Compounded Annually at: Approx. actual rate of: | Actual rate. | 5% | 6% | Remarks. |
|---|---|---|---|---|---|---|---|---|---|
| 27 | Cambridge, Mass., Water-works. | | | | | | | | |
| | *a.* Stony Brook Supply..... | 4 | 1887 | $538 757 | 3.75% | 7.1% | 9.6% | 11.6% | |
| | *b.* Payson Park Reservoir.. | 5 | 1897 | 685 151 | 3.75% | 7.6% | 10.0% | 12.2% | |
| | *c.* Hobbs Brook Reservoir.. | 4 | 1897 | 993 244 | 3.75% | 5.4% | 7.1% | 8.5% | |
| | Totals and Averages | | | $2 217 132 | 3.75% | 6.5% | 8.5% | 10.4% | |
| 28 | New Bedford, Mass., Lake Quittacus Water-works........... | 7 | 1899 | 1 256 688 | 3.66% | 9.2% | 12.7% | 15.4% | |
| 29 | Portland, Me., Water District New 14.3 mile 42-in. cast-iron pipe line from Sebago Lake Basin to City................. | 4 | 1912 | 811 201 | 4.0% | 4.1% | 5.1% | 6.2% | |
| 30 | Springfield, Mass., Little River Supply. Storage reservoir dam, intake reservoir tunnel, slow sand filters, steel pipe line and distributing reservoir.... | 3 | 1909 | 1 455 684 | 3.5% | 2.8% | 4.1% | 4.9%* | |
| 31 | Metropolitan Water-works † (Boston)...................... | Various periods | 1895 to 1906 | $23 600 000 | 3.10% | 9.42% | ...... | ...... | |
| | | | | $25 497 927 | 3.16% | 10.3% | ...... | ...... | |
| | | | | $25 497 927‡ | 3.5% | 16.8% | 25. | 31.0% | Hazen, Metcalf. Metcalf with interest allowed on distribution pipe system to 1906, instead for only 6 months as above. |
| 32 | New Orleans Water-works...... | 6 | 1909 | 7 518 868 | 4.0% | 8.3% | 10.6% | 12.7% | |
| 33 | Waterville, Maine, *et al.* Kennebec Water District—New Supply........................ | 1 | 1905 | 252 154 | 3.57% | 2.1% | 2.9% | 3.5% | 7 months period. Emergency new supply. |
| 34 | Chicago, Ill., Water-works Southwest Land and Lake Tunnel System.............. | 8 | 1911 | 3 249 754 | 4% | 14.1% | 17.9% | 21.8% | |
| | | 8 | 1911 | 3 307 285 | 4% | 14.6% | 18.5% | 22.6% | Incl. $57 580.95 item of cost in 1902. |

* Actual period 4 years, excluding preliminary investigations. Emergency case. 1910 payments of $306 457 excluded as plant in service.

† See also Nos. 7 to 11 for different structures.

‡ Excludes acquisition of old works, $14 059 964; administration, $262 606; stock, $70 878.

(*e*).—Interest During Construction.—(*Continued*).

Summary of Analyses by Leonard Metcalf, M. Am. Soc. C. E.

| No. | Locality and character of works. | Period of years. | Date of completion, December 31st. | Reported total cost. | Accrued Interest During Construction to Completion of Structure with Compound Interest at: Assumed actual rate of: | Actual. | 5% | 6% | Remarks. |
|---|---|---|---|---|---|---|---|---|---|
| 14 | Denver Union Water Company. Cheesman Dam..... ......... | 7 | 1905 | $1 229 641 | 5% | 18.7 | 22.8 | 27.0 | Excluding land and rents. Four years (net time) were consumed in investigation, surveys and planning, correspondence with Government, acquiring rights, etc.; and 6.5 years in construction, during a total period of 12 years from inception to completion. |
| 15 | Cincinnati Water-Works: Intake, pumping station, filter plant, conduit, etc..... | 11 | 1907 | 13 329 348 | 3.75% | 15.8 | 21.2 | 26.0% | |
| 16 | 1905–1914 Los Angeles Aqueduct.......................... | 9 | 1914 | $24 526 868* | 4.5% | 20% + | (Actual) | .......... | *Excluding interest. |
| 17 | 1914 Pacific Gas and Electric Company. Spaulding Dam.. | 1 + | 1914 | 4 283 359 | .......... | 6.8% | (Actual) | .......... | A rush job. |
| | | | | Structural Costs only | | | | | |
| 18 | — Big Creek...................... | .... | ........ | $11 339 308 | .......... | 8.86% | (Actual) | .......... | Quoted by Pacific Gas and Electric Company Exhibit 15, Cases 477 and 550, before California Railroad Commission. |
| 19 | — Big Bend permanent dams.. | .... | ........ | 522 437 | .......... | 3.68 | .......... | .......... | |
| 20 | — Big Bend extension.......... | .... | ........ | 476 928 | .......... | 7.10 | .......... | .......... | |
| 21 | — Kaweah No. 3................. | .... | ........ | 511 560 | .......... | 7.38 | .......... | .......... | |
| 22 | — Kern Canyon.................. | .... | ........ | 340 685 | .......... | 7.21 | .......... | .......... | |
| 23 | — Los Angeles Aqueduct....... | .... | ........ | 18 840 344 | .......... | 19.40 | .......... | .......... | |
| 24 | — San Joaquin.................. | .... | ........ | 1 563 485 | .......... | 6.0 | .......... | .......... | |
| 25 | — Tule River.................... | .... | ........ | 649 096 | .......... | 9.96 | .......... | .......... | |
| 26 | — Spaulding drum development.......................... | .... | ........ | 3 498 894 | .......... | 6.8 | .......... | .......... | |

### (*f*).—Engineering on Railways.

From Various Sources.

(1).—Actual Cost. Milwaukee, Sparta and Northwestern Railway (Chicago and Northwestern) 1910.

| District. | Miles. | Total cost of construction. | Total cost per mile. | Total cost of engineering. | Cost of engineering per mile of road. | Engineering. Percentage of road. |
|---|---|---|---|---|---|---|
| Milwaukee to Clyman Junction | 52.97 | $5 412 246.85 | $102 175.70 | $108 428.21 | $2 046.97 | 2.0 |
| Butler Yard | ...... | 891 707.04 | .......... | 7 654.47 | ........ | 0.86 |
| Clyman to Necedah | 89.44 | 5 396 907.95 | 60 342.22 | 165 008.89 | 1 844.91 | 3.06 |
| Friendship Yard | ...... | 364 559.21 | .......... | 4 410.19 | ........ | 1.21 |
| Wyeville to Sparta | 23.02 | 1 430 448.99 | 62 139.40 | 46 721.13 | 2 029.59 | 3.27 |
| Wyeville Yard | ...... | 339 005.89 | .......... | 4 581.87 | ........ | 1.38 |
| Necedah to Wyeville (Reconstruction of old road) | 13.05 | 238 186.32 | 18 251.82 | 3 535.18 | 270.89 | 1.49 |
| Average cost, entire line | 165.43 | $12 701 161.94 | $76 775.65 | $336 804.76 | $2 034.72 | 2.65 |
| | | 13 834 925.93 Cost, including right of way. | 83 630.09 | .......... | ........ | 2.44 |

(2).—Cases Presented to Interstate Commerce Commission in Argument and Brief, September 1st, 1915.

| Name. | State. | Miles. | Percentage. |
|---|---|---|---|
| Kansas City Southern | Missouri | 3.4 | 2.6 |
| Chicago and Northwestern | Wisconsin | 178.0 | 2.8 |
| Illinois Central | Kentucky | 32.0 | 1.3 |
| Sunset Central | Texas | 47.0 | 2.8 |
| Union Pacific | Nebraska | 26.0 | 2.5 |
| Southern Pacific | California | 143.0 | 2.5 |

"The total investment for these 4 390 miles of western lines was $126 711 214, exclusive of land, equipment, and overhead charge; the charge to engineering was $5 193 116, the construction cost per mile being less than $29 000.

"The East reports 42 cases (exclusive of the Pennsylvania Tunnel and Terminal Company and the New York Central developments in New York). The mileage covered is 817 miles, or but 19% of the length reported by the West, the total investment, excluding land, equipment, and overhead, being $132 528 399, or $5 817 185 more than the 4 390 miles reported by the western lines.

"The engineering cost for these 817 miles was $7 677 089, or 5.79%, an increase in rate of 41% over the western lines. The average cost per mile was $162 239, or 5.6 times the amount per mile expended on the western roads reporting."

| Name. | State. | Miles. | Percentage. |
|---|---|---|---|
| Boston and Maine | New Hampshire | 7.5 | 1.7 |
| Pennsylvania Lines West | | 6.0 | 2.2 |
| Western Maryland | | 86.0 | 2.6 |
| Delaware and Hudson | | 14.0 | 2.2 |
| New York, Ontario and Western | | 54.0 | 2.4 |
| New York Central | | 11.0 | 2.0 |
| Chicago and Eastern Illinois | Illinois | 62.0 | 1.8 |
| Chicago and Eastern Illinois | Illinois | 63.0 | 2.6 |

"The New York Central and Hudson River Railroad Company, the Barge Canal, and five cities spent $256 800 000 (including land), of which $133 000 000 was expended outside the City of New York. The engineering expense on this work that was charged amounted to 7.6% on all items of construction, including land.

"The Pennsylvania Tunnel and Terminal Company, with 11.81 miles of first track and 69.42 miles in all, cost $111 701 889, exclusive of equipment, taxes, interest, or overhead charges other than engineering; of which amount, $20 679 950 was for land, or 18.5% of the total. The charged cost of engineering was 7.2% of the cost, excluding land and equipment.

"The South reports 30 cases, covering 1 638 miles, with an investment, including land, but excluding equipment and overhead, of $78 546 665. The charged cost of engineering on these 1 638 miles was $3 492 248, or 4.4%, including land."

(*g*).—LAND.—OVERHEAD CHARGES.—STATEMENTS SHOWING SOME EXPENSES OF RAILROAD COMPANIES IN THE ACQUISITION OF LAND.—(*Continued*).

| Owning company. | Acquired for: | Total purchase price, exclusive of expense of acquisition. | Expense of acquisition. | Percentage of purchase price. | Remarks. |
|---|---|---|---|---|---|
| | *Brought forward* | $1 622 863 | $95 861 | | |
| Delaware, Lackawanna and Western Railroad | Cut-off, 't. Morris, N. Y., to Slateford, Pa. | 237 566 | 43 172 | 18.17 | |
| Hastings and Northwestern Railroad (Union Pacific Railway Company) | New line, Nebr. | 211 862 | 9 493 | 4.48 | |
| Lehigh Valley Railroad Co. | Irvington Branch | 129 945 | 2 676 | 2.06 | |
| | Lehigh and Lake Branch | 1 290 588 | 35 718 | 2.77 | |
| | Hays Creek Branch | 25 163 | 3 113 | 12.37 | Of the percentage of purchase price, 7.25 represents services of attorneys. |
| | Seneca Falls Extension | 39 998 | 1 243 | 3.12 | |
| Long Island Railroad Company | Elimination of grade crossing at Bushwick Junction, Long Island | 84 475 | 3 384 | 4.01 | |
| | Elimination of grade crossing at Queens, Long Island | 73 083 | 4 067 | 5.56 | |
| | Additional right of way and station grounds, Brooklyn | 712 812 | 38 473 | 5.4 | Expense ratio as to property purchased, 4⅛%; condemned, 24 per cent. |
| | Passenger coach yard, Jamaica, Long Island | 234 356 | 13 371 | 5.7 | Expense ratio as to property condemned, 4.2%; condemned, 15 per cent. |
| Marion and Southern Railroad (Seaboard Air Line) | Extension in South Carolina | 12 856 | 701 | 5.45 | |
| Milwaukee, St. Paul and Sault Ste. Marie Railway | Fordville-Drake Line, N. Dak. | 237 942 | 27 403 | 11.51 | High expense due to the great number of small purchases—about 160—averaging less than $250. Legal expense was more than 80% of total. |
| | Ceylon-Downing cut-off | 54 248 | 5 494 | 10.13 | |
| Norfolk and Western Railway | Main line widening, Nottaway Co., Va. | 35 775 | 13 761 | 38.40 | |
| | Main line widening, Dinwiddie Co., Va. | 8 483 | 1 154 | 13.16 | |
| | Main line widening, Isle of Wight Co., Va. | 2 529 | 687 | 27.18 | |
| Northern Pacific | Passenger coach yard, St. Paul, Minn. | 411 508 | 3 553 | 0.86 | Amount paid for improvements $317 437. |
| | Bear Creek Branch, Mont. | 114 758 | 5 384 | 4.70 | |
| | Knife River Branch, N. Dak. | 44 566 | 9 259 | 20.77 | |
| | New passenger line, Minneapolis | 490 023 | 28 192 | 5.80 | |
| New York, New Haven and Hartford Railroad | Elimination of grade crossings, Boston, Mass. | 120 410 | 4 568 | 3.70 | Includes expenses of trial attorneys, but not of purchasing, title, or general office, which were not separately charged. |
| | Elimination of grade crossings, Olneyville, Providence, R. I. | 126 106 | 4 070 | 3.20 | |
| | *Totals carried forward* | $6 321 910 | $354 797 | | |

(*g*).—LAND.—OVERHEAD CHARGES.—STATEMENTS SHOWING SOME EXPENSES OF RAILROAD COMPANIES IN THE ACQUISITION OF LAND.

Statement of Thomas W. Hulme, General Secretary, Presidents' Conference Committee, at Conference between Division of Valuation, Members of State Commissions, and Representatives of the Presidents' Conference Committee of the Railroads, May 28th, 1915.

| Owning company. | Acquired for: | Total purchase price, exclusive of expense of acquisition. | Expense of acquisition. | Percentage of purchase price. | Remarks. |
|---|---|---|---|---|---|
| Baltimore and Ohio Railroad | Produce yard, Baltimore, Md. | $206 724 | $10 186 | 4.90% | Expense ratio as to property purchased, 3¼ per cent. Expense ratio as to property condemned, 16.7 per cent. |
| Bessemer and Lake Erie Railroad | Kremis to Osgood, Pa. (K. O. Line) | 63 211 | 3 299 | 5.22 | $342.96 included in purchase price would more properly be included in expense; and *vice versa* as to $892.56. To cover salaries, $500 has been used. |
| Boston and Maine Railroad | Mechanicville, N, Y., transfer yard | 28 185 | 336 | 1.30 | |
| | Newport, Vt., freight yard | 18 450 | 812 | 4.40 | |
| | Salem, Mass., freight yard | 6 920 | 554 | 8 0 | |
| | Guyandot Valley Extension, W. Va. | 28 400 | 1 848 | 6.51 | |
| Chesapeake and Ohio Railroad | Buffalo Creek Branch, W. Va. | 44 095 | 1 925 | 4.37 | |
| | Big Sandy Branch Extension, Ky. | 236 953 | 25 091 | 10.59 | Acquired in 1902. |
| | Coal River Branch, W. Va. | 43 931 | 2 355 | 5.36 | |
| | Yard at Peru, Ind. | 11 850 | 964 | 8.14 | Much negotiation required. |
| | Gravel pit near Peru, Ind. | 9 970 | 612 | 6.14 | |
| | Terminal purposes at East End, Binghamton, N. Y. | 282 129 | 4 775 | 1.69 | |
| Delaware and Hudson | Delanson to Schenactady, N. Y. | 212 195 | 32 498 | 15.25 | Of this ratio 12.46% represents expenses incurred in condemnation proceedings. |
| | Terminal purposes at Schenectady, N. Y. | 346 750 | 8 896 | 2.56 | |
| | Yard and cold storage plant, Glenville, N. Y. | 83 100 | 1 710 | 2.06 | |
| | *Totals carried forward* | $1 622 863 | $95 861 | | |

(*g*).—LAND.—OVERHEAD CHARGES.—STATEMENTS SHOWING SOME EXPENSES OF RAILROAD COMPANIES IN THE ACQUISITION OF LAND.—(*Continued*).

| Owning company. | Acquired for: | Total purchase price, exclusive of expense of acquisition. | Expense of acquisition. | Percentage of purchase price. | Remarks. |
|---|---|---|---|---|---|
| | *Brought forward* | $6 321 910 | $354 797 | | |
| Pennsylvania Railroad | Darby Creek Line | 1 522 400 | 73 610 | 4.84 | Of the 4.84% representing expense of acquisition, 1% engineering expense on account of surveys on present and former locations. |
| Philadelphia, Baltimore and Washington Railroad (Pennsylvania Railroad Company | Glynden to Lovell | 62 896 | 6 628 | 10.53 | The large number of properties, and widely scattered ownership, made it necessary to consume much time in traveling. |
| | Additional tracks, Newport to Pencader, Del. | 32 205 | 3 687 | 11.45 | Difficult negotiations and widely scattered ownership, taken in connection with the small total purchase price, made the expense ratio rather high. |
| Pomeroy Belt Railway (Hocking Valley Railway Company.) | Extension near Pomeroy, Ohio | 14 317 | 926 | 6.47 | |
| Quemahoning Branch Railroad (Baltimore and Ohio Railroad Company.) | Husband line | 56 750 | 3 464 | 6.10 | In the purchase price, as given, are included $4 859 paid for damages to crops, change of roads, privilege of wasting material, etc. If this amount be excluded from the purchase price, the expense ratio will be 6.60 per cent. |
| Vandalia (Pennsylvania Railroad West.) | Harmony to Terre Haute, Ind. | 112 756 | 4 390 | 3.89 | |
| | Eagle Creek Connection. Ind. | 26 018 | 1 596 | 6.18 | |
| Winston-Salem South Bound Railway Company (Atlantic Coast Line.) | Tar Branch Terminal Line | 240 483 | 3 558 | 1.48 | The figures submitted show an expense ratio of 1.73%, based on a total purchase price of $205 299. The accompanying statement, however, shows that the purchase price should be $240 483. Hence the ratio of 1.48 per cent. |
| | Totals | $8 889 730 | $452 656 | 5.4% | |

(*h*).—ANALYSIS OF CERTAIN LAND PURCHASERS BY RAILROADS.

*Details of passenger coach yard purchase, St. Paul.*

There were no condemnations, all tracts being bought at private sale.

There were no severance damages, the entire tract being bought in each case.

The law requires the Tax Commission to determine ratio of assumed valuation to real value, and the "assessment method" is based on actual assessment affected by this ratio.

| | Before the identity of the purchaser was known. | After the identity of the purchaser was known. |
|---|---|---|
| Area purchased | 581 120 sq. ft. | 388 931 sq. ft. |
| Total purchase price | $94 000.00 | $269 700 |
| Deduct amount paid for improvements | 11 083.33 | 69 132 |
| Amount paid for land | $82 916.67 | $200 568 |
| Price per square foot paid for naked land | 0.143 | 0.558 |
| Total naked land value determined by assessment method | 44 535.00 | 49 536 |
| Naked land value, assessment method, per square foot | 0.077 | 0.138 |
| Excess paid over value by the assessment method | $38 381.67 | $151 032 |

Summarized, the above figures show:

| | | |
|---|---|---|
| Naked land value determined by assessment method. | $94 071.00 | 23% |
| Paid for improvements | 128 023.47 | 31% |
| Paid, in excess of value, by assessment method | 189 413.66 | 46% |
| Paid for severance damages | 0 | 0 |
| Expenses of acquisition | $411 508.13 | 100% |

The actual expense of the Right-of-Way Department was $3 553.20, or only 0.86% of the total purchase price. There were 21 parcels acquired and two others secured by exchange of site and removal of buildings, making 23 owners dealt with. Expense per parcel, $154.50.

*Details of Knife River Branch, N. Dak., Northern Pacific Railway.*

| | Mercer Co., N. Dak. | Dunn Co., N. Dak. |
|---|---|---|
| Valuation of naked land, made by real estate men and bankers, not by employees. Per acre | $22.07 | $22.13 |
| Total | $15 709.42 | $12 065.12 |
| Total number of deeds | 86 | 63 |
| Total acres conveyed | 784.97 | 615.55 |
| Average acres per transfer | 9.12 | 9.77 |
| Total consideration | 27 460. | 17 106. |
| Average consideration per acre | 35.00 | 27.80 |
| Total number of considerations | 4 | None |
| Acres condemned | 70.83 | |
| Award in condemnation cases | 1 966.52 | |
| Average per acre, award | 27.70 | |

Summary.—Both Counties.

| | | |
|---|---|---|
| Total naked land value.......... | $27 774.54 | 62.32% |
| All other elements of value...... | 16 791.89 | 37.68% |
| Total purchase price............ | $44 566.43 | 100% |
| Expense of acquisition......... | 9 258.79 | |
| Ratio of expense of acquisition to purchase price............ | 20.77 | |

Expenses of Acquisition.

| | Total. | Average per deed. | Average per acre. |
|---|---|---|---|
| Services and expenses of right-of-way agents. | $6 790.75 | $46.19 | $4.85 |
| Services and expenses of attorneys......... | 705.69 | 4.80 | 0.50 |
| Cost of abstracts and opinions of title....... | 1 227.80 | 8.35 | 0.93 |
| Cost of recording deeds.................. | 329.55 | 2.24 | 0.23 |
| Automobile hire.......................... | 205.00 | 1.40 | 0.15 |
| | $9 258.79 | $62.98 | $6.66 |

This case affords an illustration of the extremely favorable cost of acquisition. The line was desired by the people. There was but little contest. Two of the condemnation cases were to acquire land owned by the State of North Dakota, the other two were against land companies. Every individual owner was settled with, yet, in spite of these facts, the land cost was $27 774.54, and "other elements" plus cost of acquiring amounted to $26 050.68, or 93.8% of the bare land price.

*Details of Bear Creek Branch, Northern Pacific Railway.*

The Bear Creek Branch, built in an irrigated valley under high cultivation, paralleling within 2 miles an existing road, was not desired by the farmers.

The assessed value, based on sales of 5 916 acres in 1913-14, was 25.3% of sales. On this the assessor's statement that 25% is a fair average is accepted for part of the district, but 20% was used on part.

Condemnation occurred in seven cases, in each case the award was divided into (*a*) actual value of strip taken, (*b*) damages to remainder.

| | | |
|---|---|---|
| Area taken by condemnation.... | 23.57 acres | |
| Award for strips taken......... | $3 624.00 | 13.2% |
| Award for damages............. | 23 930.00 | 86.8% |
| Total award ............... | $27 554.00 | 100% |

It is to be noted that the award for strips averaged $153.75 per acre, as against an actual land value, based on assessment method and opinion, of $59.85 per acre plus $20.00 for growing crops. This award was $73.90 in excess of actual value.

The total purchases were as follows:

| | | | |
|---|---|---|---|
| Bridger to Belfry...... | 155.78 acres at $59.85 | $9 323.43 | |
| Belfry to Bear Creek.. | 104.26 " " 25.15 | 2 622.06 | |
| | 260.04 " | | |
| Bear Creek town lots............................ | | 2 206.08 | |
| Total naked land value................. | | $14 151.65 | 12.33% |
| All other elements of value............. | | 100 606.60 | 87.67% |
| | | $114 758.28 | 100.00% |
| Expense of acquisition...................... | | $5 384.35 | |
| Ratio of expense of acquisition to total purchase price ........................... | | 4.7% | |

Details of costs of acquisition, 56 parcels, 260 acres plus 59 town lots.

| | Total. | Per parcel. | Per acre.* |
|---|---|---|---|
| Abstracts of title.............. | $725.25 | $12.95 | $2.70 |
| Recording deeds............... | 111.40 | 2.00 | 0.41 |
| Legal ............................ | 168.25 | 3.00 | 0.62 |
| Salaries and expenses right-of-way men .................. | 4 379.45 | 78.20 | 16.22 |
| | $5 384.35 | $96.15 | $19.95 |

*Details of Land Purchases, Norfolk and Western Railway, Nottoway Co., Va. Main Line and Second Track Construction.*

| | Acres. | Parcels. | Average acres per parcel. | Total considera-tion. | Naked land value. | Other elements of value. |
|---|---|---|---|---|---|---|
| Town land........... | 13.40 | ... | ..... | $14 006 | $5 870 | $8 136 |
| Country land.......... | 87.86 | ... | ..... | 21 769 | 10 068 | 11 701 |
| | 101.26 | 163 | 0.633 | $35 775 | $15 938 | $19 837 |

| | | |
|---|---|---|
| Averages: Town land, average per acre, naked land. | $438.06 | 41.9% |
| " " " other considerations ............................ | 607.16 | 58.1% |
| Total.................... | $1 045.22 | 100% |

| | |
|---|---|
| The expenses of acquisition.......................... | $13 761.06 |
| Ratio of expense to cost of acquisition..................... | 38 4/10% |
| Legal expense included in cost of acquisition............ | $11 186.28 |
| Cost of acquisition, other than legal expense............. | $2 574.78 |
| Ratio of expenses, other than legal, to cost............. | 7 2/10% |

* The 59 town lots computed as 10 acres.

| | |
|---|---|
| Total expense of acquisition, per acre | $135.90 |
| Total expense of acquisition, per parcel | 84.40 |
| Expense, exclusive of land, per acre | 25.43 |
| Expense, exclusive of land, per parcel | 16.10 |

The elements in "Other elements of value" were $2 105 for moving buildings, $2 040 for severance damages, and the remainder was consequential damages.

| | Acres. | Parcels. | Total consideration. | Naked land value. | Other elements of value. |
|---|---|---|---|---|---|
| Dinwiddie County | 22.54 | 23 | $8 482 66 | $3 245 | $5 238 |
| Isle of Wight County | 5.436 | 12 | 2 529 | 580 | 1 949 |

Expense of acquisition, Dinwiddie Co................ $1 154.33, or 13.16% of purchase price.

Expense of acquisition, Isle of Wight Co............ 687.32, or 27.18% of purchase price.

| | |
|---|---|
| Expense per acre, Dinwiddie Co | 51.00 |
| " " " Isle of Wight Co | 126.44 |
| " " parcel, Dinwiddie Co | 50.19 |
| " " " Isle of Wight Co | 52.87 |

*Investigations of the California Commission.*

In the proceedings to determine the valuation of the Petaluma and Santa Rosa Railway, the finding was written by Commissioner Shelen. In his discussion of right of way, he states (Decision No. 2348—Case No. 145—Publ. Serv. Reports, 1915 C 752):

"The records of the engineering department contain the analysis of some 1 140 miles of recently constructed railroad rights of way in California, the total cost of which is $8 211 632.65, and the market value of the land for ordinary purposes, at the time of purchase, was $6 293 862.89, which would show a multiple of 1.30. The incidental expense in connection with the acquisition of 844 miles of this right of way was $648 329, which is 9.14% of the amount paid to the grantors. Practically 88% of the costs of lands was for property located within incorporated city limits, and the multiple on this classification was approximately 1.25."

## (*i*).—Cost of Acquiring and Assembling Lands for Reservoirs and Rights of Way.

Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Year. | Locality and purpose. | Area, in acres. | Total cost, excluding overhead and interest. | Cost per acre. | Excess of actual cost over appraised value, excluding overhead and interest. | Excess over assessment. | Authority. |
|---|---|---|---|---|---|---|---|
| 1912 | Kendall Reservoir, Worcester, Mass., Water-Works.......... | 407.84 | $3 917* | $95.80* | 59% | ........ | City Engr. McClure. *Including some buildings. |
| 1915 | Fitchburg, Mass., sewage disposal area.......... | 115.8 | 24 323 | .......... | 6% | ........ | Engr. Hartwell. One item $7 193 ( — 23.3 acres) settled by Court procedure; excess cost, 23 per cent. |
| 1914 | Newport, R. I., Water-Works valuation proceedings....... | .......... | .......... | .......... | 118% | ........ | Excess of Company's witnesses' estimate based on actual cost. over City's witnesses. |
| | By condemnation............. | 30.56 | .......... | 712.00 | 137% | ........ | 140.72 acres of adjoining land sold short time before for $250 per acre. Fair value estimated, $300. |
| 1899 | (Boston) Metropolitan Water-Works in Wachusett Reservoir.......... | 4 772.00 | 1 182 000 | 248.00 | 69% | ........ | F. P. Stearns, Chf. Engr., Am. Soc. C. E. Val. Com. Rept., December 1st, 1913, p. 56—Appraisers liberal estimate was for land $244 000 plus buildings $453 000; total, $697 000. Owing to compulsory feature the Board added in its estimate 50% excess plus 15% overhead, exclusive of interest. The actual cost slightly exceeded this. |
| | Individual parcels, included in the above: | | | | | | |
| | (*a*) Clinton Settlements excluding mill damages.... | .......... | 157 044 | .......... | 54% | 97% | |
| | (*b*) Boylston do.......... | .......... | 155 439 | .......... | 36% | 85% | |
| | (*c*) West Boylston do....... | .......... | 555 910 | .......... | 18.5% | 115% | |
| | (*d*) Sterling Settlements... | .......... | 20 104 | .......... | 7% | 141% | Wachusett Reservoir Data: |
| | (*e*) Settlements in Clinton, Boylston, and West Boylston.......... | .......... | 868 393 | .......... | 23% | 106% | Water surface.......... 4 135 acres<br>Contents.......... 64 968 000 000 gal.<br>Shore line, excluding islands. 37 miles, |
| | (*f*) Settlements in Clinton, Boylston, West Boylston, and Sterling.......... | .......... | 888 497 | .......... | 22.5% | 106% | Required removal of 6 large mills, 8 school houses, 4 churches, 360 dwelling houses. Old road discontinued, 19.25 miles; new road built, 11.8 miles. Cost of January 1st, 1913: |
| | (*g*) Right of way acquired for relocation of Mass. Central R. R. (18 parcels) | 29.36 | 20 074 | .......... | 32% | ........ | Wachusett Dam.......... $2 386 375.23<br>North Dike.......... 792 264.68<br>South Dike.......... 187 075.55 |
| | (*h*) Right of way for Weston Aqueduct (70)......... | 197.35 | 110 640 | .......... | 77% | ........ | Removal of soil.......... 2 536 612.66<br>Relocation of railroad.......... 881 872.45 |
| | (*i*) Ditto (9 parcels)........ | 242.34 | 60 811 | .......... | — 14% | ........ | Roads and bridges.......... 547 867.76 |
| | (*j*) Court Awards: | | | | | | Real estate taken.......... 3 240 398.41 |
| | Central Massachusetts Railroad.......... | 1.18 | 917 | .......... | 116% | ........ | Damages, business, real estate not taken.......... 532 247.07 |
| | Weston Aqueduct.......... | 0.36 | 1 147 | .......... | 2 190% | ........ | Legal and expert, except real estate.......... 8 547.92 |
| | | 3.81 | 1 785 | .......... | 297% | ........ | $11 063 261.73 |
| | | 5.64 | 550 | .......... | 57% | ........ | |
| | | 17.17 | 25 270 | .......... | 747% | ........ | |
| | | 1.96 | 1 764 | .......... | 320% | ........ | |
| | | 6.84 | 9 000 | .......... | 41% | ........ | |
| | | 5.02 | 800 | .......... | 16% | ........ | |
| | | 41.98 | $41 233.38 | .......... | 252% | ........ | |

(i).—Cost of Acquiring and Assembling Lands for Reservoirs and Rights of Way.—(*Continued.*)
Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Location and purpose. | Area, in acres. | Total cost, excluding overhead and interest. | Cost per acre. | Excess of actual cost over appraised value, excluding overhead and interest. | Authority. |
|---|---|---|---|---|---|
| Kensico Reservoir | 3 182 | Award<br>Expenses, 53.17%. | $742<br>404<br>$1 146 | 100% | All but 2 acres settled for. Area, 1.33 times water surface.<br>Hazen estimates fair value of this property during last 5 years at $800 to $1000 per acre. |
| Hill View Reservoir | 163 | Award<br>Expenses, 44.01%. | $8 796<br>3 871<br>$12 667 | | |
| Southern Aqueduct | 957 | Award<br>Expenses, 39.66%. | $1 987<br>787<br>$2 774 | | Includes East View filter site. |
| City Aqueduct (Part owned by City) | 166 taken<br>90 proceedings | Award<br>Expenses, 15.03%. | $6 585<br>962<br>$7 547 | | |
| Springfield, Mass., Water-Works | | | | 50% | Hazen estimates excess cost of lands bought at about 50% "a remarkable record and one that will be very seldom equalled in city business." |

(*i*).—COST OF ACQUIRING AND ASSEMBLING LANDS FOR RESERVOIRS AND RIGHTS OF WAY.—(*Continued.*)
Compiled by Leonard Metcalf, M. Am. Soc. C. E.

| Year. | Locality and purpose. | Area, in acres. | Total cost, excluding overhead and interest. | Cost per acre. | Excess of actual cost over appraised value, excluding overhead and interest. | Authority. |
|---|---|---|---|---|---|---|
| | Fitchburg, Mass., Water-Works, Wachusett Lake | 118.88 | $8 050 | $68 | 211% | Valued by three appraisers for City. |
| 1905.... | Cambridge, Mass., Right of way for Hobbs Brook Conduit | 2.34 | 29 639 | 9 873 | 28% | Excluding one parcel, the excess was 65%, including buildings. |
| | (Boston) Metropolitan Water-works: | | | | | |
| | (*a*) Wachusett Reservoir | 7 946 | 2 959 300* | 372* | | *Including mill settlements and water diversion damages. |
| | (*b*) " Water-shed | 556 | 148 890 | 268 | | |
| | (*c*) " Aqueduct | 309 | 81 677 | 264 | | |
| | (*d*) Sudbury Reservoir | 2 772 | 658 994 | 237 | | |
| | (*e*) " Water-shed | 391 | 16 522 | 42 | | |
| | (*f*) Weston Aqueduct | 480 | 188 925 | 393 | | |
| 1914.... | New York, Catskill Water Supply | 21 827 | 10 669 946 | 509 | ............ | Based on Board of Water Supply figures, December 31st, 1914, and estimate of 1905 by J. Waldo Smith, M. Am. Soc. C. E. Litigation on value of 345 acres still pending. |
| | Interest | | 1 648 801 | 78 | | |
| | Indirect and consequential damages | | 294 801 | 14 | | |
| | Fees, legal and sundry expenses | | 4 188 795† | 200 | ............ | †No. 39. 26% overhead excluding interest. See analysis below. (p. 7 of Hazen's letter.) |
| | | | $16 808 344 | $801 | | |
| | Excluding interest | | ............ | $723 | | |
| | Ashokan Reservoir | 15 221 | Award | $243 | ............ | Litigation on 316 acres still pending. Area 1.87 times water surface. |
| | | | Expenses, 61.41%. | 155 | | |
| | | | | $398 | 100% | A little more than twice Engr. Sears' estimate. |
| | Northern Aqueduct | 1 637 | Award | $470 | ............ | Settlements on all but 37 acres. |
| | | | Expenses, 91.08%. | 428 | | |
| | | | | $898 | 800% or more. | Hazen estimates value of these lands at between $50 and $100 per acre. |

## APPENDIX IV.

### Records of Abandoned Property.

The Committee calls the attention of engineers and administrators of public utility properties to the desirability of keeping as clear and complete a record as posible of the property abandoned by the corporation, annually and in aggregate amount.

Such records will be of great service to Courts and commissions, in assisting them to determine the annual depreciation allowance which should fairly be made for any public utility property, in the light of its past history, as well as of an intelligent forecast of its probable future.

It is obvious that such records—when used as a check on, or when coupled with, an estimate of the accrued depreciation on the existing property—will reduce substantially the element of uncertainty which must attend all such computations, based, as they must always be, on probable future, as well as on past, conditions.

As an example of such records within the water-works field, and of their applicability, the following table is cited, embodying the results of a continuous effort to collect such data, during a period of 15 years, more or less, on the part of Allen Hazen and Leonard Metcalf, Members, Am. Soc. C. E.

It seems highly probable that, by keeping such records in detail, there will be developed, in the course of time, sufficient information to admit of the setting aside of the fair annual depreciation allowance on a basis of percentage-of-gross-revenue of the operating company, a method which would have much advantage, on account of its simplicity of accounting, as well as for other reasons.

Inasmuch as this table is submitted for suggestive purposes only, and not for direct application, it is unnecessary to record here a description of the various plants, or the methods of determining the figures contained in the record, though it may be added that these figures have been taken in large measure from authoritative sources, such as Court, commission, or agreed, findings.

### DEPRECIATION RECORDS OF SOME OLD WATER-WORKS.

Assembled and Analyzed by Allen Hazen and Leonard Metcalf, Members, Am. Soc. C. E., December 3d, 1915.

| | Portland, Me., Water District. | Racine, Wis. | Denver Union Water Company, Denver, Colo. | Pennsylvania Water Company, Wilkinsburg, Pa. | Spring Valley Water Company, San Francisco, Cal. | Indianapolis Water Company, Indianapolis, Ind. |
|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
| Date of record | 1907 | 1913 | 1913 | 1911 | 1913 | 1915 |
| Authority | Hazen and Metcalf | Hazen | Metcalf | Metcalf | Hazen and Metcalf | Metcalf |
| Population | 78 200 | 38 002 | 242 000 | 90 000 | 453 000 | 270 000 |
| Whole period of operation, in years | 39 | 26 | 43 | 23 | 55 | 45 |
| Average age of investment, in years | 18 | 16 | 20 | 12 | 26 | 14.2 |
| Gross revenue during whole operating period | $6 605 000 | $1 343 000 | $22 000 000 | $3 923 000 | $76 968 000 | $13 972 000 |
| Gross reproduction cost of structures | $2 594 000 | $1 219 000 | $10 411 438 | $3 128 389 | $25 126 000 | $8 156 000 |
| Abandoned structures | 137 000 | 19 000 | 1 420 505 | 250 000 | 3 420 000 | 883 000 |
| Total (including overhead and interest during construction) | $2 731 000 | $1 238 000 | $11 831 943 | $3 378 389 | $28 546 000 | $9 039 000 |
| Accrued depreciation, existing structures | $498 000 | $70 600 | $1 320 970 | $271 700 | $3 496 000 L. M | $748 000 |
| Accrued depreciation, in abandoned structures | 137 000 | 19 100 | 1 420 505† | 250 000 | 3 420 000 | 883 000 |
| Total depreciation, in amount | $633 000 | $89 700 | $2 741 475 | $521 700 | $6 916 000 | $1 631 000 |
| " " in percentage | 23.2% | 7.24% | 23.2% | 15.4% | 24.2% | 18.1% |
| " " in percentage on average net reproduction cost | ........ | ........ | ........ | ........ | ........ | 19.8% |
| Average annual straight-line depreciation, in percentage, based on: | | | | | | |
| (a) Gross reproduction cost and abandoned structures | 1.29% | 0.45% | 1.16% | 1.29% | 0.93%‡ | 1.27% |
| (b) Net or depreciated reproduction | 1.46% | 0.47% | 1.31% | 1.39% | 1.06%§ | 1.39% |
| (c) Gross revenue during whole operating period | 9.57% | 6.7% | 12.4% | 8.00%* | 8.45% | 11.7% |
| (d) Per capita (total depreciation divided by present population multiplied by average age) | $0.45 | $0.15 | $0.57 | $0.48 | $0.59 | $0.42 |

| Average of 6 plants. | Average of 5 plants, excluding Racine. |
|---|---|
| 1.07% | 1.19% |
| 1.18% | 1.32% |
| 9.3% | 10.0% |
| 0.44% | 0.50% |

* Figures 7.52%; 8% believed to be a fairer figure, however (Metcalf).

† Including Sullivan Reservoir.

‡ On basis of original cost of structures of $21 306 000, a straight-line depreciation rate of 1.25% was found to yield $6 774 000; depreciation allowance (including abandoned property) as of December 31st, 1913. Note that $\frac{\$21\ 306\ 000}{28\ 546\ 000} \times 1.25\% = 0.93$ per cent., an approximate check.

§ On Hazen's depreciation, 0.99 per cent.

Development Division of Some Big Water-Works

[illegible] December 31, 1916.

| [illegible] | Indianapolis Water Company, Indianapolis, Ind. | Spring Valley Water Company, San Francisco, Cal. | [illegible] Water Company, [illegible] | Denver Union Water Company, Denver, Colo. | [illegible], Wis. | [illegible] |
|---|---|---|---|---|---|---|
| (1) | (7) | (6) | (5) | (4) | (3) | (2) |
| [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |

[illegible]

# MEMOIRS OF DECEASED MEMBERS

**NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.**

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## EDWARD MANNING BIGELOW, M. Am. Soc. C. E.*

DIED DECEMBER 6TH, 1916.

Edward Manning Bigelow was born in Pittsburgh, Pa., on November 6th, 1850, being one of five children born to Edward M. and Mary Steel Bigelow. He received his early education in the public schools and after graduation entered the Western University of Pennsylvania (now the University of Pittsburgh) where he studied civil engineering.

Upon leaving college he entered the service of the City of Pittsburgh in a subordinate position on the staff of the City Engineer, which position he held until 1874 when he was promoted to engineer in charge of certain street construction, working under a Commission which was authorized by special legislative enactment. Subsequently, the laws authorizing this work were declared unconstitutional, and Mr. Bigelow left the service of the City for a short time. He was re-employed by the City and served as Assistant Engineer in charge of surveys and the location of streets from 1876 to 1878, and in charge of construction from 1878 to 1882.

On January 9th, 1882, he was unanimously elected City Engineer by Council, which position he held until 1885, and from 1885 to 1888 he was City Engineer and Commissioner of Highways. At this time the City Charter was changed, the Department of Public Works being created, and Mr. Bigelow was then appointed Director of the Department of Public Works, which office he held for two full terms of three years each.

On June 1st, 1911, Governor John K. Tener of Pennsylvania appointed Mr. Bigelow to the office of State Commissioner of Highways, which office he held until April, 1915. Mr. Bigelow was honored by a re-appointment to the office of Director of the Department of Public Works of Pittsburgh just prior to his death.

Mr. Bigelow's entire life was devoted to municipal improvements. He was an engineer of broad vision and foresight; his plans for improvements were always comprehensive, and provided well for the future. Under his supervision many miles of Pittsburgh's streets and sewers were laid out and constructed, including the boulevards. His

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* Memoir prepared by N. S. Sprague, M. Am. Soc. C. E.

greatest achievement, and the one in which he took the most pride, was the development of Highland and Schenley Parks. It was also under his administration that work on the water purification plant was started.

Mr. Bigelow was an honest, tireless, and conscientious worker, and more than ordinary credit is due him from the people of Pittsburgh when it is considered that many of the improvements which he inaugurated and carried through to successful completion were consummated in the face of much criticism and abuse. The appreciation of his services in the development of the parks was manifest by the erection of a bronze statue placed at the entrance of Schenley Park, the cost of which was defrayed by public subscription. He was known as the "Father of the Parks".

The same characteristic plan on a large and comprehensive scale was displayed by Mr. Bigelow during his tenure of office as Commissioner of Highways of the State of Pennsylvania. Under his direction plans were made for the improvement of a comprehensive system of highways traversing the State in anticipation of securing funds for their improvement. Many of the State highways were improved, and much credit is due to him for the creditable work performed with the limited funds at his disposal.

Even during periods when Mr. Bigelow was not connected with the government of Pittsburgh, his interest in city affairs never ceased; and his genius for planning and energy in advocating improvements for the benefit of the city were always manifest.

Aside from Mr. Bigelow's professional career, he was a man of affairs and took an active interest in financial, commercial, and social activities. He was a Trustee of the Carnegie Institute of Technology from the time of its founding, and took an active part in the development of this institution. He was President of the Liberty National Bank, the Liberty Savings Bank, and the Homewood Cemetery. He was also a Trustee of Andrew Carnegie's benefactions, the Carnegie Library and Institute, Carnegie Hero Fund Commission, the Institute of Technology, and also of the Schenley Memorial Commission. He was a member of, and also took an active part in, the East Liberty Presbyterian Church, and his time and talents were given freely in the raising of funds and for other purposes connected with this church.

Upon Mr. Bigelow's death, the Council of the City of Pittsburgh paid due respect by calling a special meeting of that body for the purpose of honoring the memory of the late Director of the Department of Public Works, and the following resolution was passed, which perhaps is the best expression of the high regard and esteem in which his services to the City of Pittsburgh are held by this community:

"*Whereas,* Almighty God, in His infinite wisdom, has seen fit to remove from our midst one of Pittsburgh's most distinguished citizens; and

"*Whereas,* Edward Manning Bigelow was a man of stainless integrity, of superb courage, and of great intellectual force; a man who always walked in the highway of right; and in disaster stood erect; a man to whom defeat was but a spur to further effort; a man who believed in the loyalty to man, in the sovereignty of the citizen, and in the matchless greatness of this grand City; a man who gave freely of his talents and genius to the up-building of the City of Pittsburgh, who served the City for many years with a self-sacrificing devotion, and whose master hand is indelibly impressed upon every section of the City; and

*Whereas,* As he lived he died, with that tenacity of purpose which characterized his life; while yet in love with life and raptured with the world he bravely answered the call of the Master of the Universe and passed to his eternal home beyond the utmost reach of human harm or help. He has left with us his wealth of thought and deed—the memory of a brave, imperious, honest man, who bowed alone to death; therefore be it

*Resolved,* that a copy of these resolutions be spared upon the Minutes of Council of Pittsburgh, and that a copy be sent to his bereaved widow."

In addition to the foregoing resolution, many tributes of respect were paid to Mr. Bigelow's ability as a citizen and public officer by men prominently identified with the financial, commercial, and professional affairs of the City.

Mr. Bigelow was elected a Member of the American Society of Civil Engineers on December 4th, 1889.

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## WILLIAM BYRD KING, M. Am. Soc. C. E.*

DIED OCTOBER 11TH, 1915.

Mr. King was born in Orange County, Virginia, on December 29th, 1848, and was a direct descendant of Colonel William Byrd, who founded Richmond in 1733.

His early education was through tutors and private neighborhood schools. Later, he took academic work at Locust-Dale College and some engineering courses at the University of Virginia.

In 1873 Mr. King went to Texas, where he engaged in land surveying, principally the demarcation of railway land grants in the Far West. He continued this pioneer work, under all sorts of hardships

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

and dangers, until 1879, when he was appointed Assistant Engineer on Construction of the Texas and Pacific Railway, and later of the Fort Worth and Denver City Railway, and the Missouri, Kansas and Texas Railway, in Texas.

In 1884 Mr. King was appointed City Engineer of Fort Worth, serving until 1889, when he was appointed City Engineer of Waco, serving until 1891. During his incumbency of these two offices he planned street and sewerage systems that were basic for both municipalities.

After completing his work at Waco, Mr. King was called to the Chief Engineership of the Fort Worth and Rio Grande Railway, now a part of the Frisco System. Later, he was elected Vice-President and General Manager of the Frisco System in Texas, which position he filled with distinction until Armour and Swift established their great packeries and stockyards in Fort Worth, in 1902, when he was made President of the Fort Worth Belt Railway, and Vice-President and General Manager of the "Armour-Swift" Stockyards.

For several years Mr. King was a sufferer from arterio-sclerosis, and in 1915 was retired, with consultation duties only, on full salary. For perhaps two years before this he had suffered from the insomnia characteristic of his malady to the extent that 10 or 12 hours' sleep a week was his gauge of rest. In spite of this, he was regularly at his desk, the suave, courteous, directing head of the busiest cattlemart of the Southwest.

In addition to his other engagements, he found time during 1913 to act as a member of the Board of Engineers in charge of Fort Worth's new water supply. His work on that Board helped to bring to light and eradicate one of the most insidious pieces of municipal graft ever known in the South.

During several of the quite strenuous Board meetings on this matter, he was obliged to retire to his physician's office and submit to copious bleeding, in order to keep his blood pressure within living bounds. Little things like letting out part of his heart's blood in order to stand the strain of serving the public did not seem to "cut any figure".

The writer knew "Byrd" King, with increasing intimacy, from 1891 until his death. He was the kind of man we all want really to have as a friend. An Engineer of the Tredgold type. A typical Southern gentleman? No. Byrd King's courtly manners marked him as the son of many generations of Virginia Cavaliers, but his gentleness went far beyond any qualification as to South or North: his innate, never spoken, but always to be depended on, rule of action was, *noblesse oblige.*

William Byrd King was a world gentleman.

Mr. King was elected a Member of the American Society of Civil Engineers on October 7th, 1896.

## HENRY GORDON STOTT, M. Am. Soc. C. E.*

DIED JANUARY 15TH, 1917.

Henry Gordon Stott was a native of the Orkney Islands, Scotland, where he was born on May 13th, 1866, the son of the Rev. David and Elizabeth Jane (Dibblee) Scott. After a thorough grounding in the fundamentals, by his father and elementary school instructors, he was enrolled as a student at the Watson Collegiate School, Edinburgh. On leaving this institution he entered the College of Arts and Sciences at Glasgow, and began a course in mechanical engineering and electricity, being graduated in 1885. In the year previous he had entered the employ of the Electric Illuminating Company, of Glasgow. Shortly after his graduation he was made Assistant Electrician on board the Steamship *Minia,* belonging to the Anglo-American Telegraph Company. The next 4½ years saw him engaged with those duties, during the course of which he saw much service in connection with repairs to the cable lines of that company. During this period he undertook a number of experiments that resulted in the introduction of improved methods of handling cable repairs. He was also identified with the "duplexing" of the United States Cable Company's main cable (2 750 knots), the longest duplex cable in the world.

In 1889 Mr. Stott was made Assistant Engineer of the Brush Electric Engineering Company's plant at Bournemouth, England. In the following year he accepted a position with Hammond and Company as Assistant Engineer in the construction of an underground cable and power plant at Madrid, Spain. He remained there until 1891, coming to the United States in that year to construct an underground cable and conduit system for the Buffalo Light and Power Company (now the Buffalo General Electric Company). This work was completed with a degree of success that reflected very great credit on Mr. Stott. As a result he was named Engineer of the company, and during the next 10 years was one of the most active figures in the industrial progress of Buffalo. During this period he designed and executed some notable construction work, including a power-plant on Wilkeson Street, Buffalo.

His efforts attracted wide attention, and in 1901 he was appointed Superintendent of Motive Power of the Interborough Rapid Transit Company, New York City, a position which he filled with signal success. At the time he took up these duties the Interborough had not yet been organized, the company having the title of the Manhattan Railway Company. The post to which Mr. Stott was called had just been created, and it devolved upon him to organize the operating force,

*Memoir prepared by W. S. Finlay, Jr., Esq.

in connection with which he completed the 74th Street power-plant of the company, and various sub-stations and transmission lines.

When the Manhattan System was amalgamated with the Interborough, in 1904, Mr. Stott was invited to retain his office with the new corporation. He accepted, and immediately took over supervision of the construction of the power-plant on 59th Street. Since that time he had been constantly in charge of the design, construction, and operation of the power generating stations and the distributing system of the Interborough, which comprehends both the subway, elevated, and surface lines of New York City.

The plans for the electric power system of the new subway lines were developed under his supervision, and the work has progressed so far and bears so strongly the stamp of his work that, when completed, it will be a monument to him.

Mr. Stott was a firm believer in co-operation among engineers, through the agency of engineering societies. He was elected President of the American Institute of Electrical Engineers for the term 1907-1908, Vice-President of the American Society of Mechanical Engineers for the term 1912-1914, Director of the American Society of Civil Engineers in 1911, and was Vice-President and Trustee of the United Engineering Society at the time of his death. Up to the last, Mr. Stott was a recognized power in the American Institute of Electrical Engineers, and was a member of the Standards Committee, the Committee on Development of Water-Power, the United States National Committee of the International Electrotechnical Commission, the Power Stations Committee, the Public Policy Committee, the Edison Medal Committee, the Committee on Economics of Electric Service, and was one of the Institute's representatives on the Joint Committee on the Metric System, of which he was an ardent advocate.

As a result of his unusually wide experience and extended research, Mr. Stott was called on often to contribute papers to the various engineering societies. He was especially well known for his minute analysis of enginering problems. Among the many papers written by him on this subject are "The Conversion and Distribution of Received Currents", "Power Plant Economics", "Notes on the Cost of Power", "Steam Pipe Covering and Its Relation to Station Economy", "Tests of a 15 000-Kilowatt Steam Engine Turbine Unit", "Power Plant Design and Operation" (a series), etc.

Mr. Stott was a remarkable figure in the engineering world, because he was in the front rank of both electrical and mechanical engineers, because, in both branches of the art, he was a master of theory and practice, and because, with these technical qualifications, he combined a rare executive ability, a power of inspiring the confidence of his employees, and of bringing out the best that was in the men who worked for him.

Mr. Stott's activities were not confined to engineering matters. He early became a citizen of the United States and served for 5 years in the 74th Regiment of the National Guard of New York State.

He was also an active member of the Protestant Episcopal Church, being a communicant of St. Paul's Church, of New Rochelle, in the affairs of which he was deeply interested.

Mr. Stott was married on July 22d, 1894, to Miss Anna Mitchell, of Belfast, Ireland, who, with their two children, a son and a daughter, survives him. He died at his home, in New Rochelle, N. Y., on January 15th, 1917, after an illness of many months.

Mr. Stott was elected a Member of the American Society of Civil Engineers on July 1st, 1908.

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## JAMES JEROME HILL, F. Am. Soc. C. E.*

DIED MAY 29TH, 1916.

The death of James Jerome Hill, at St. Paul, Minn., on May 29th, 1916, removed from the railway, banking, and industrial life of the United States one of the most forceful and beneficent figures of his generation. Through the reflex action of his marvelous mind, as demonstrated in the many activities directly affecting the progress and welfare of the world, he became an international character, and will live in the future, in the influence and example of the sound policies and great works that his genius created during the busy years of his life.

In the vast circle of his friends, and among his business associates who knew him as he was, the demonstration of grief at his death was sincere and heartfelt, and the knowledge that such a personal and public loss was inevitable, in the course of Nature, made the sorrow no less poignant.

To condense even the main features of his history and life work within the limits of this memoir would be an impossibility, but, as a Fellow of the American Society of Civil Engineers, it is fitting that some record of his career and activities, and of the encouragement and co-ordination that he gave to the civil engineers who carried out his great works, should be made in the annals of the Society.

Mr. Hill was born near Guelph, Ontario, in the Dominion of Canada, on September 16th, 1838. His parents, James Hill and Anne (Dunbar) Hill, were of Scotch and Irish extraction, his father following the usual occupations of a farmer in a new country. His school education was obtained at Rockwood Academy, an institution

* Memoir prepared by Ralph Budd, W. L. Darling, and John F. Stevens, Members, Am. Soc. C. E.

of his home neighborhood, conducted by a Quaker schoolmaster, who exercised a strong influence on him and directed his reading and studies. His father died when he was fifteen years old, and he then took his share in the maintenance of the family by working in a country store near-by. His ambition, however, could not long be satisfied with such a narrow field, and three years later, in the summer of 1856, after visiting several of the larger cities of the United States, he settled in St. Paul, Minn.

As a young man without friends or resources, he found employment as a clerk on the river front, then the center of the business life of the little frontier settlement. He was first with the Dubuque and St. Paul Packet Company, then with the Davidson line of steamboats, remaining with that concern until he was appointed Agent of the Northwestern Packet Company in 1865. In 1867 he established a general fuel and transportation business on his own account, and in 1869 he became the head of the firm of Hill, Griggs and Company, engaged in the same enterprises. During all these years, by constant study of historical facts, human nature, and conditions, he laid the foundation of a business experience which came to include a knowledge of all the essential facts of the trade of the city and the country back of it, and began to estimate properly the wonderful resources of the latter, the possibilities of which had not even dawned upon our most far-sighted men.

The fuel business, even at that time, was important, and was closely allied with the restricted transportation facilities of the Northwest; but, during this period, Mr. Hill had become interested in the trade between the United States and Fort Garry, as Winnipeg was then called, and which was carried by way of St. Paul. To handle this business, he established, in 1870, the Red River Transportation Company, and in 1872 he started the first regular line of service between St. Paul and Fort Garry. The knowledge he thus gained of the vast latent resources of that country crystalized into one of the dominant ideas of his existence, and one which, extended and amplified beyond his wildest dreams, became an accomplished fact long before the close of his life. So that, as the result of his prophetic vision and tireless energy, an immense section of our country, which had been considered—if at all—of little value, has become a very potential factor in the production of the wealth of the world, and the homes of thousands of the best citizens of which the United States can justly boast. It was on one of his many trips, made in connection with this transportation business, that he met Donald A. Smith, afterward Lord Strathcona, with whom he was to be so closely and successfully associated in his first railroad enterprise.

There came to a troubled life in the Fifties, under the stimulus of legislation in Minnesota, a railway company called the St. Paul and

Pacific, which name alone signified ambition, without knowledge or resources. It controlled valuable terminal lands in St. Paul, and had partly built a line, ultimately intended to reach the Canadian boundary at St. Vincent; but the road had failed, and had gone into the hands of a Receiver in 1873. Its bonds were held in Holland, by parties who were not willing to abandon their investment, nor had the foresight or nerve to invest more to salvage the wreck. Mr. Hill alone saw the possibilities of the road, and what a factor it could be made toward the development of the country, an empire then unsettled and overlooked. It was a ready forged but crude instrument, which, to be adequate for his purpose, must be controlled, properly financed, and made effective, all of which appeared to be a hopelessly visionary project, excepting to his master mind.

Its total debt was nearly \$33 000 000, an enormous sum for those times, and the very size of this debt would have staggered a less optimistic and forceful character. He believed that he could secure, and bent all his energies toward securing, control of the property, and his faith was justified. The co-operation of Donald A. Smith, and of George Stephen (afterward Lord Mountstephen) was gained, and, together with Norman W. Kittson, who had had experience of the country in acting for the Hudson Bay Company, they formed a syndicate to buy the defaulted bonds of the companies covered by the St. Paul and Pacific Railroad Company.

The detailed story of the next few years reads more like a fairy tale than a relation of the sober facts of history. These four men pledged every thing they had in the world to obtain even money enough to secure an option on the bonds; for men of affairs in New York and London regarded the whole proposition as chimerical. Besides the securing of an option on the bonds, money had to be provided to finish uncompleted lines, within the time limit set by the Legislature, in order to avoid forfeiture; and all of this labor fell upon Mr. Hill, as did also the financing necessary to make good the agreements with the bondholders, until such time as the railroad could become a producer of net revenue, and it was not until 1878 that it could be truthfully said that matters really began to take proper shape.

In 1879 the properties were reorganized as the St. Paul, Minneapolis and Manitoba Railway Company, with necessary lines in operation, an organization created, the proper spirit infused into the enterprise, and a matured plan for the future mapped out. The system then comprised 555 miles of constructed road, and 102 miles under construction. From this time began its steady, never-ceasing expansion, covering the Northwestern country with new lines and branches, to be followed closely by immigration and settlement, and making an equally steady progress towards Montana, and the ultimate goal of the Pacific Ocean, which object had been in Mr. Hill's mind from an early day.

In 1888 the road reached Butte, and in 1893, the Pacific Extension touched the shores of Puget Sound at Seattle, Everett, and Bellingham, realizing the transcontinental idea. A line had long before been completed to the Great Lakes, and a steamship company established to give an eastern outlet.

Mr. Hill was for two years during this period interested in the planning and construction of the main line of the Canadian Pacific Railway, from the plains of Manitoba west to the Pacific Ocean, being Managing Director of that company; but in 1883 he gave up direct connection with that enterprise to devote his entire time to the welfare of his own marvelously developing properties.

He opened trade with the Orient, establishing a line of transpacific ships to develop and handle this new traffic. Locally, wherever his lines touched, new industries sprang up; immigration and settlement, fostered by him as a personal matter, followed the rails with a rapidity hitherto unprecedented in the history of the United States.

In 1890 the various railroad enterprises which were offshoots of the original plan were all compacted into one company, the present Great Northern System. The great machine was pushed to completion, and grew bigger and better from year to year, until it stands to-day, in character and plan, as a marvel of transportation efficiency.

Mr. Hill was General Manager of the company from its beginning, became Vice-President in 1881, and President in 1882, which office he held and administered until he resigned it to one of his sons, just 25 years later. During all these years, as well as to the end of his life, his was the guiding hand, and his the inspiring vision and controlling mind that have made the Great Northern Railway not only the most notable long-distance transportation machine in the world, but also the greatest single factor in the material development of an empire of incalculable wealth of resources. This system, as the embodiment of the plans, ideas, and acts of one man, alone constitutes a fitting monument to the clear vision and untiring genius of James J. Hill.

As a student of railway economics, Mr. Hill had early known that the measure of success for a road was to be found in its ability to move traffic at the lowest practicable cost. In freight movement, this meant light gradients, with heavy power and large-capacity cars, with consequent maximum train loading. In this field he was preeminently the pioneer in American railroad practice. In the planning and construction of his lines, the cardinal principle, which he insisted must be followed, was that a dollar saved in construction is saved but once, but that a dollar saved in operation, is saved every time the wheels go over the track, and he saw to it that his engineers carried this principle out in every particular. Either by personal inspection or from reliable reports, he knew practically the resources and conditions of every mile of territory into which he projected his lines. He was naturally a

great engineer, and always appreciated and gave credit where credit was due, to the different engineers whom he trusted to plan and execute the details of his projects. He quickly recognized merit and ability in the personnel of his engineering staff, and neither seniority nor rank influenced him in the least in preferment or promotions. He was equally quick to criticize and condemn, whenever his usually unerring judgment so dictated, and his attention to details together with his dynamic force of character kept every one of his staff up to the highest point of individual efficiency. But every engineer, however humble his rank, knew that, like Napoleon's soldiers, he carried a baton in his baggage, and the success and preferment of many engineers, not only of the Great Northern, but of other systems, were due, not only to Mr. Hill's forceful example, but also to his direct personal interest in their welfare. Whenever he gave his confidence and support, he gave freely and fully. Results were what he demanded, and nothing pleased him more than to have an engineer leave the beaten track and find a new and better way to reach such results, and in this way individual initiative was encouraged. The Engineering Profession owes a large debt of gratitude to Mr. Hill, as some who were intimately associated with him for many years know full well, and appreciate.

As evidence of the great respect he had for the Profession, let his own words speak for themselves. In an address delivered before a gathering of Civil Engineers, at Minneapolis, Minn., on January 10th, 1908, he said:

"Yours is a great profession, following as it does the oldest of all the arts. When the pyramids were built there were engineers who mastered problems calling for great engineering skill. Relics of still older civilizations appear from time to time on the sites of cities whose names forgotten tell that the engineer was there also; and that he was, then as now, a leader of thought in matters of great practical service to the common good. The name of your occupation is significant, related as it is to the word 'genius' and implying the union of human ability with that diviner spark which kindles knowledge to a brighter flame.

"The engineer is, indeed, in no mean sense a creator. Not only does he provide for the needs of mankind those practical utilities on which civilization is based, but he brings into being structures, combinations, possibilities of dead matter and natural energy in relations new to men. He is inventor as well as artisan. Many of the modern wonders of the world are the thoughts of his brain as well as the works of his hands. The subjection of Niagara to our daily wants, the redemption of an ancient country by the damming of the Nile, great bridges such as span the Firth of Forth and our own East River at New York, tunnels that carry traffic under the estuary of the Severn and the peaks of the Simplon pass, tubes that enable railways to avoid waters too wide to be bridged safely, canals that cut away land barriers, the modern steamship that has cheapened and quickened ocean transit,

the factories that have revolutionized industry and the marvelous plants which are now using the waterfalls of Norway to enrich the earth by the manufacture of fertilizers drawn from the nitrogen of the air—all these are conquests of the engineer's ability and bear tribute to him as the author of heretofore unthought of things. Across every chapter of the story of human development is written the symbol of some great engineer's creative mind."

The financial record of Mr. Hill's labors sufficiently attests to the sagacity of his ideas and their practical execution. While to superficial comment, it may be said that his individual pecuniary reward was great, the fact remains that he was never a money seeker for personal benefit. His whole ambition was to be a faithful custodian of the interests committed to his care, and no charge that he ever failed to make his work good can justly be made against him. His ability to interest capital—domestic and foreign—in any project, or in line with any plan he set forth, was well known. He proved his faith and words by his acts, and such a course made him almost unique in the history of railroad finance in the United States.

As a characteristic act, his handling of the Great Northern iron ore interests may be mentioned. In 1899 he purchased an insignificant logging road, in northern Minnesota, which owned some lands. These lands contained iron ore, but in unknown quantities. At that time the vast deposits of such ore in that section had not been demonstrated, and his purchase involved a business risk in which he did not care to engage his stockholders. So the property was acquired as a personal investment. A company was formed to hold and develop the lands, which was done in the succeeding years, and when their immense value had been fully proven, they were all turned over and distributed to the holders of Great Northern stock at cost. What the present and potential value of the lands is no one knows, but it is a safe judgment that they alone are worth more than the personal estate which Mr. Hill acquired during his lifetime.

During all the busy years he gave to the creation and building up of the great system of railway, not a cent of salary was ever drawn by him from it, he preferring to take his compensation in common with the stockholders, from the increased value of the properties. Unlike other transcontinental lines, the Great Northern received no land grant, or other aid from the Federal Government, excepting the grant originally made to the St. Paul and Pacific Company in Minnesota. Its securities were marketable because they had value behind them, and from the further fact that the interests of the road were in the hands of a man who always had made, and was believed to be always able to make, his word good.

In 1901, the Great Northern and the Northern Pacific Railways jointly purchased the stock of the Chicago, Burlington and Quincy

Railroad Company, thus giving to his other lines access to the enormous producing and consuming territory of the upper Mississippi Valley, a territory which for extent and variety of wealth-producing area probably cannot be duplicated in the world.

Mr. Hill had planned to place all the properties, in which his stockholders and himself were interested, in a position to be secure from future speculative attacks, and to ensure the harmony which efficiency demands, by uniting them in the Northern Securities Company. That company was dissolved by the decision of a divided Court, but the management of the separate companies, so far, has remained successful and harmonious.

Although the chief activities of Mr. Hill's life were expended in the development of railways, he did work enough along other lines to have more than sufficed to keep an ordinary man fully occupied. The keynote of such work was his desire to raise the average level of prosperity, of intelligence, and of public spirit. Undoubtedly, the greatest pleasure he took in his work was in the efforts he made and the success he achieved in improving agriculture and in increasing its profits. He led the farmer away from the one-crop idea, and taught him to diversify his industry by stock raising. He imported, at large personal expense, stock of the most profitable strains, and distributed it, after practical demonstration on his own large experimental farm, among the most intelligent farmers of the Northwest, most of whom he knew personally. This practice he continued through thirty years, and the splendid condition of the live stock interest of that section to-day is largely the result of his labors.

He taught soil restoration and rotation of crops; set up a laboratory for soil analysis, and established demonstration plots, on farms along the line of the railway, to demonstrate to the farmers, by object lessons, what startling results could be obtained by scientific fertilization and cultivation. He was always ready to talk to a group of farmers, and gave no small part of his life's forces, through his later years, to the effort to persuade them to better methods.

He was the first to advocate the conservation of natural resources on a large scale. The memorable address which he delivered in 1906 created a profound impression, and ran through many editions and languages. He made heavy demands on his business time and physical resources to comply with requests to address bodies of representative men assembled to discuss economic and scientific problems, but he regarded this as a form of public service which it was his duty to perform.

Irrigation, waterways, all forms of internal improvement, banking and other forms of finance, the theory of credit, and the virtue of thrift, every aspect of railroad operation, regulation, and improvement, almost every practical human activity received his study and profited

by the conclusion of his wonderfully analytic and almost unerring mind. Some of the more important of these were grouped in his volume of economical and sociological discussions, entitled "Highways of Progress", published in 1910. It is amazing to observe, in that collection, the range of his thought, the wealth of his information, and the validity of his conclusions.

Among the larger interests of his later years was the acquisition of the two banks in St. Paul, and their consolidation under the name of the First National, to which was joined the Northwestern Trust Company, and all placed under the immediate control of one of his sons. This great financial institution is intended to be, not merely a clearing house for the enormous financial interests of the properties in which he was concerned, but an agency for liberating the Northwest from outside control, and for increasing and stabilizing its prosperity. It has been and is a power in advancing the interests of the farmers and helping them to surer and larger returns. Until a few days before his death, he was as busy as he had been for years before, with plans for the upbuilding of the country and the improvement of the common lot.

He was a patron and lover of the arts. His collection of pictures is known at home and abroad by its excellence, and is the result of his judgment in selection. He was a connoisseur of precious stones, and his investments in these were marked by his usual good business judgment. He was an omnivorous reader, and, being possessed of a remarkable memory, had accumulated a wealth of detailed information upon a wide range of subjects, such as few men of his age, or any age or generation, ever possessed. His philanthropy did not stop with words, for he was extremely generous, in a practical, material way. He gave impartially to many educational institutions, his only condition being that the schools that received his aid should be conducted under the auspices of some religious denomination.

His private benefactions were constant and great; they were made with care and discrimination, and he avoided publicity in them to the last degree.

For many years, during the earlier part of his career, he cannot be said to have attracted, excepting among his own group of business associates and capitalists, the more than passing notice of the then leaders of thought and finance. He never attempted to attract such notice by spectacular staging of any of his enterprises. His sayings and doings were rather looked upon by most eastern men with a mild commiseration; but, as his system of railways developed, and the results of his so-called theories began to prove their value, and to be seen in many respects to be the very basic foundation of the prosperity of the railways of the country, they were copied by other systems, and, from train haul to accounting, the Great Northern was for years a school

at which railway men were glad to learn; and many graduates from that school, trained by practical experience in its methods, can be found holding high positions in railway service, in all parts of the United States. During the last decade of his life, there was no one whose voice, in general economics and financial matters, was listened to with more respect than that of Mr. Hill. The judgment of no one man in this country, perhaps, counted for so much in determining American financiers to take up the first loan asked by the Allies. He thought its acceptance essential to our own business life and prosperity, and that his reasons were sound and his conclusions well taken, is amply proven by subsequent developments.

Though his plans were well wrought out, and the results he achieved came from thorough preparation, he could strike with lightning quickness and terrific force, whenever occasion demanded. He was plain of speech, simple and sincere of thought, but a giant of power when roused to effort by obstacles that had to be overcome, and he was—first, last, and all the time—an American. The men on his railroads recognized his character and ability, and through all classes and ranks there ran a spirit of cheerful loyalty that could only have been evoked by a man of his wonderful personality. Among the many who were privileged to know him intimately, particularly the old-timers, who had been with him in the long-gone-by days of his early struggles, the feeling was one of deep and sincere affection, and everywhere, over the whole nation and in other countries, the general sentiment of respect and sorrow at his death, as a personal and public loss, has been deep and wide.

It is a safe prediction that the great record of what he achieved will insure increased recognition as time goes on. Yale University conferred upon him the degree of LL.D. in 1910. The Hill Professorship of Transportation was established at Harvard not long ago, with an endowment contributed by seventy-four friends who were delighted to do him this honor.

He was a man of quiet domestic tastes, and his serenest hours were those spent in his beautiful home at St. Paul, where his friends were always welcome, and from which they always came away feeling that they had been privileged to share in the happiness of an ideal American fireside. Throughout all his years, he demonstrated in his own life that a strict adherence to moral laws was a true attribute of greatness.

Much of the planning of his construction work in all lines was done in the seclusion of his home, to which he habitually drew men of all classes for conference and instructions, and his library there was the workshop in which was produced many of his masterpieces.

On August 19th, 1867, he was married to Miss Mary Theresa Mehegan, of St. Paul, and his widow, three sons, and six daughters survive him.

Volumes of anecdotes could be written to illustrate his manner of dealing with various situations. Through them all runs the strong vein of humor which was so marked a feature of his character. Above all, he asked for independence in judgment by his subordinates, and no surer path to his disfavor could be found than to endeavor to forestall his opinion, and recommend accordingly. Though he was a man of strong convictions, even in matters of minute detail, he was always open to argument, and ready to receive and weigh suggestions, and to give them consideration. Once a decision was made, however, he expected speed and economy in the prosecution of the project in hand, and was content with nothing less than the best results. Take him all in all, to one who understood in a measure the salient features of his individuality, and realized the broad view-point from which he regarded every subject, he was an eminently satisfactory man to be associated with, and such association was a liberal education.

After Mr. Hill's death, hundreds of messages of sympathy poured in upon the family from all parts of the country, from lands across the sea, from people of the greatest diversity of condition. It was a spontaneous tribute which showed in what esteem he was held by high and low alike. Corporations, associations, municipal bodies, cities, and States, united to regret him and to do honor to his memory. It is impossible to reproduce here any considerable portion of these tributes; but an extract, as of date June 3d, 1916, from the minutes of the Board of Directors of the Great Northern Railway Company, the creation of which was his greatest life work, shows to what degree he was appreciated by his close associates:

"*Resolved,* That in the death of James J. Hill, the Great Northern Railway Company laments the loss of the man who brought it into being, who made it an instrument for the development of this country, who led it with unerring wisdom, who foresaw and safeguarded its future, and to whom it owes all that it has been and all it hopes to be."

James Jerome Hill was elected a Fellow of the American Society of Civil Engineers on January 10th, 1889.

## PAPERS IN THIS NUMBER

"UNUSUAL COFFER-DAM FOR 1 000-FOOT PIER, NEW YORK CITY." CHARLES W. STANIFORD. (To be presented Mar. 7th, 1917.)

"THE RECONSTRUCTION OF THE STONY RIVER DAM." F. W. SCHEIDENHELM. (To be presented Mar. 21st, 1917.)

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# PAPERS AND DISCUSSIONS



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# AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## MULTIPLE-ARCH DAMS ON RUSH CREEK, CALIFORNIA

BY L. R. JORGENSEN, M. AM. SOC. C. E.

TO BE PRESENTED APRIL 18TH, 1917.

### SYNOPSIS.

This paper describes the design and construction of two multiple-arch dams built during the summers of 1915 and 1916. The determination of span length is first discussed, followed by general rules for the selection of the angle subtended by the individual arches. It is pointed out how the arch slope affects the load distribution on the arches, and the stability of the buttresses. Stress diagrams for the arches and buttresses are given, most of the calculations being graphical. The increase in deformation of concrete under sustained load, and its effect on stresses due to temperature changes is brought out, and an attempt is made to show that the time factor (time interval between the occurrence of maximum and minimum temperature) can to some extent be depended on to prevent or minimize temperature cracks. To stiffen the buttresses, counterforts are used in connection with struts. Test results of the concrete as used are given. Compression specimens were made in the field, and aged there, not under laboratory conditions, but out in the open dry atmosphere. The

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

up-stream faces of the arches were covered with a 1:2 coat of cement and sand, applied with a cement gun. In conclusion some cost data are presented.

## Introduction.

During the last few years several multiple-arch dams have been built in the United States, and, thus far, have been very successful in operation. Undoubtedly, many more would have been constructed had a thorough knowledge of their design and construction been more general among dam-building engineers. There are places where rock or earth fill dams, or a combination of the two, are now built, where multiple-arch dams could have been constructed more economically, and they would have been more substantial.

Although, under ordinary conditions, a rock or earth fill dam can be constructed with a sufficient, although unknown, factor of safety, such dams are absolutely unsafe under abnormal conditions, such as when water accidentally passes over the crest. It is a well-known fact that nearly all failures have been due to this cause. Water passing over the crest of a multiple-arch dam would not destroy it, and, for a more or less limited time, would not hurt the foundation, if this was otherwise at all safe for such a structure. A multiple-arch dam requires a good foundation, as the load is concentrated on the buttresses, and settlement of these would be likely to cause the collapse of adjoining arches. Whenever the foundation is solid rock, however, a multiple-arch dam can be constructed and will be as substantial as any, and more substantial than most types. The stresses and dimensions can be calculated with great accuracy. The factor of safety of such a structure, therefore, is known within narrow limits, assuming first-class construction, and on that account, precedents should not be given so much consideration as with rock and earth fill dams. These latter cannot be subjected to calculation of stresses, and therefore have to be built mostly along lines dictated by precedents. In general, it can be said that a multiple-arch dam of small and medium height (less than 100 ft. high) will cost less to construct than a rock fill dam, especially if the latter is provided with something better than a wooden up-stream face for the water-tight cut-off. Perhaps only in rare cases would there be occasion for comparing the relative costs of a multiple-

arch and a strictly earth fill dam, because, if there is enough suitable earth to construct an earth dam, a sufficiently good foundation for a multiple-arch dam and sufficient good building material for such a dam are not likely to be found at the same place, and *vice versa*.

For a multiple-arch dam higher than, say, 130 ft., the quantity of building material required, and therefore the cost of such a structure, increases quite rapidly, due mainly to the fact that the buttresses become very large and require more bracing. Eventually, therefore, a limit of height is reached, where it will be more economical to build a single arch across the canyon, unless the latter is very wide. This limit of height for any dam can only be found by trying all the types possible of application, as the shape of the dam site has also quite an influence on the design.

## The Design of the Multiple-Arch Dam.

The first thing to be determined is the length of each individual span. Unless there should be strong reasons for using different span lengths for the several arches making up the complete structure, all spans should be the same, in order to facilitate the form work. Theoretically, the shorter the spans, the less the material required for the arches. The material required for the buttresses remains theoretically the same, no matter what length of span is chosen. Although a dam consisting of small spans takes less material than one where the spans are larger, the cost of construction may not necessarily be less. The form work becomes more extensive, and it is more difficult to place the concrete and reinforcing steel in the resulting narrow space between the form boards than in a wider space. Thin arch walls are more likely to collapse than thicker ones, and thin buttresses would require elaborate bracing in order to prevent their collapse long before their crushing strength had been reached. It is the arch that holds the water back, and therefore the water-tightness of the dam is to some extent a function of the thickness of this wall, although, to a much larger extent, it depends on the quality of the building material (concrete) used.

Taking all these facts into consideration, it may be stated that the practical and most economical span lies between the limits of 30 and 50 ft. For high dams, the economical span is near the upper limit; for low dams, it is near the lower limit. A 40-ft. span would be a

good average value for ordinary cases, and is chosen in the present instance.

The next feature to be determined is the length of the up-stream radius. It is known that the most economical arch* is the one that subtends an angle of 133½°, and that, for variations of about 10% on each side of this angle, the difference in economy is very small. For the dams to be described later the subtended central angle at the up-stream face is 120°, or, to be exact, 119° 57′. The volume of the arch has thereby been increased approximately 1% above the theoretical minimum, but, at the same time, the thickness has been increased 6%, thereby decreasing the ratio of thickness of arch to length of arch, which, for structural reasons, is a desirable feature, at least toward the crest where the thickness is small compared with the length. This also decreases the probability of percolation by decreasing the area of the wetted surface, and by increasing the thickness of the wall.

With the subtended angle (120°) and the span (40 ft.) decided, the length of the up-stream radius is calculated to be 23.1 ft. To facilitate form work, the length of this radius is kept constant from crest to foundation, except as noted later. Incidentally, this gives also the most economical arch, as the subtended angle is thereby kept practically constant. The arch is given a slope with the horizontal of 50°, in order that the water pressure, acting on the vertical projection of this slope, may tend to cut down the shearing stress on the buttresses to zero, or to some insignificant value.

The arch carries the total water load and a large part of the load due to its own weight. A preliminary arch thickness may be found by the simple formula,

$$P \times R_u = q \times t \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which $P$ equals the water pressure, in pounds per square foot; $R_u$ equals the length of the up-stream radius, in feet; $q$ equals the average stress, in pounds per square foot, of the area of the dam section under consideration; and $t$ equals the thickness of the dam, in feet, at any given horizontal elevation.†

* Further information in reference to this may be found in *Transactions*, Am. Soc. C. E., Vol. LXXVIII, Paper No. 1322, pp. 689–690.

† The thickness of any horizontal arch slice (elliptical with major axis $= \frac{R_u}{\sin. 50^\circ}$, and with minor axis $= R_u$) is made constant from abutment to abutment, and therefore the circular arch perpendicular to the slope must have its thickness increase from the crown toward the abutments.

After the thickness, $t$, has been determined, at as many points as thought necessary, say every 10 ft. apart in elevation, the weight of the arch can be calculated and the additional arch stress due to this load determined. If the total stress is found to be excessive, a new thickness will have to be chosen.

Due to the inclination of the arch, the water pressure is not distributed uniformly between the crown and the abutments on any imaginary arch slice perpendicular to the slope. As the crown is at a higher elevation, it sustains less water pressure than corresponding points at the abutments.

For the case under consideration—where the length of the upstream radius is 23.1 ft.; the subtended angle is 120°; and the inclination of the arch is 50° with the horizontal and 40° with the vertical—a point at the crown will always be

$$\frac{R_u}{2} \times \text{sin. } 40° = 11.55 \times 0.64379 = 7.424 \text{ ft.}$$

above a corresponding point at the springing line. Toward the crest this difference in elevation makes a large difference in the distribution of the load on the face of the arch, and therefore, also, in the location of the line of pressure in the arch ring. Take, for instance, an arch slice 1 ft. wide, with the middle of the crown at Elevation 5, and with the middle of this slice at the springing line at Elevation $5 + 7.424 =$ Elevation 12.424. With reservoir full to the crest, the water load at the middle point of the crown would correspond to $5 \times 62.5 = 312.5$ lb. per sq. ft., whereas the water pressure at the corresponding point at the springing line would correspond to $12.424 \times 62.5 = 776.5$ lb., or nearly 2.5 times more than at the crown.

At lower elevations this large difference (2.5 times) rapidly grows less, and becomes unimportant with crown elevations below Elevation 15. In the upper portion of the dam the radial component of the arch weight borne directly by the arch itself tends to neutralize the large difference in water pressure between the crown and the corresponding abutment points, as this weight component has its largest value at the crown, and its smallest value at the abutments. This can be shown most clearly by a graphical method, and in Figs. 1 and 2, two arch slices are drawn, each assumed to be 1 ft. wide. In Fig. 1 the middle of the crown is at Elevation 5, and then a width of 6 in. is taken on each side; and in Fig. 2 the crown is at Elevation 10. The

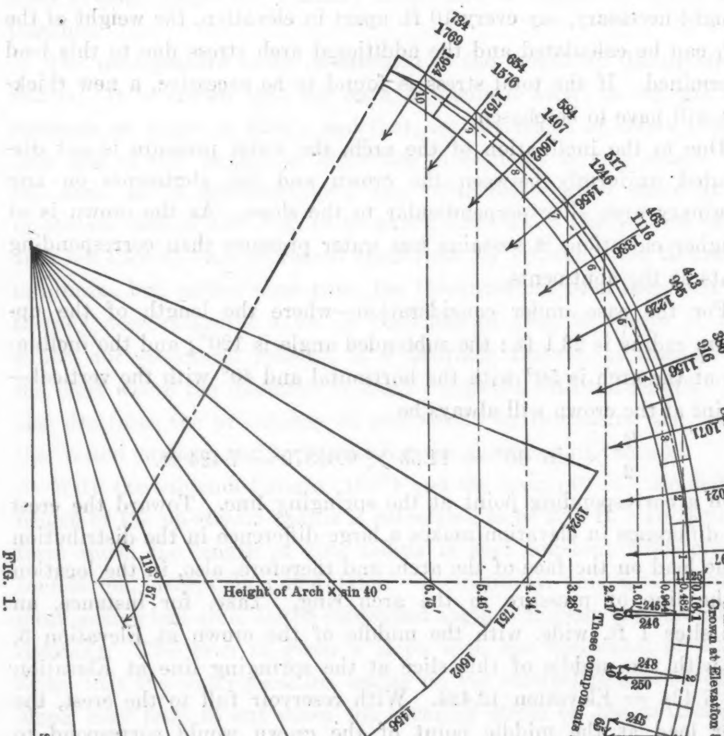
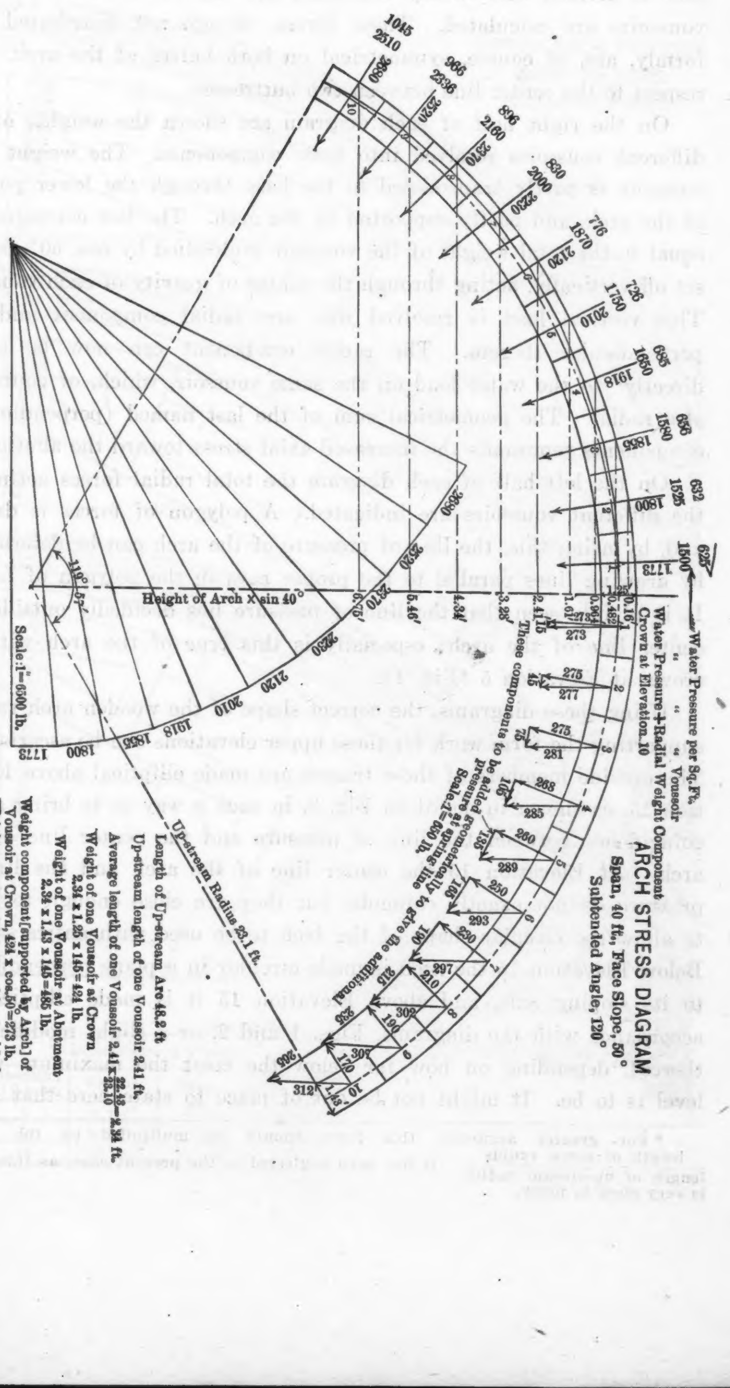

ARCH STRESS DIAGRAM
Span, 40 ft., Face Slope, 50°
Subtended angle, 120°
Water Pressure per Sq. Ft.
Water Pressure + Radial Weight Component
Crown at Elevation 5
Voussoir
These components to be added geometrically to give the additional pressure at Springing line
Scale 1"=700 lb.
Height of Arch × sin 40°
Up-stream Radius 23.1 ft.
Scale: 1"=3500 lb.

FIG. 1.



ARCH STRESS DIAGRAM
Span, 40 ft., Face Slope, 50°
Subtended Angle, 120°
Water Pressure per Sq. Ft.
" " " Voussoir
Water Pressure + Radial Weight Component
Crown at Elevation 10
These components to be added geometrically to give the additional pressure at spring line
Scale: 1" = 650 lb.
Height of Arch X sin 40°
Up-stream Radius 23.1 ft.
Scale: 1" = 6500 lb.
Length of Up-stream Arc 48.2 ft
Up-stream length of one Voussoir 2.41 ft.
Average length of one Voussoir 2.41 x 22.43/23.10 = 2.34 ft.
Weight of one Voussoir at Crown
2.34 x 1.25 x 145 = 424 lb.
Weight of one Voussoir at Abutment
2.34 x 1.43 x 145 = 485 lb.
Weight component [supported by Arch] of
Voussoir at Crown, 424 x cos. 50° = 273 lb.
Weight component [supported by Arch] of
Voussoir at Abutment, 485 x cos. 50° = 312 lb.


FIG. 2.

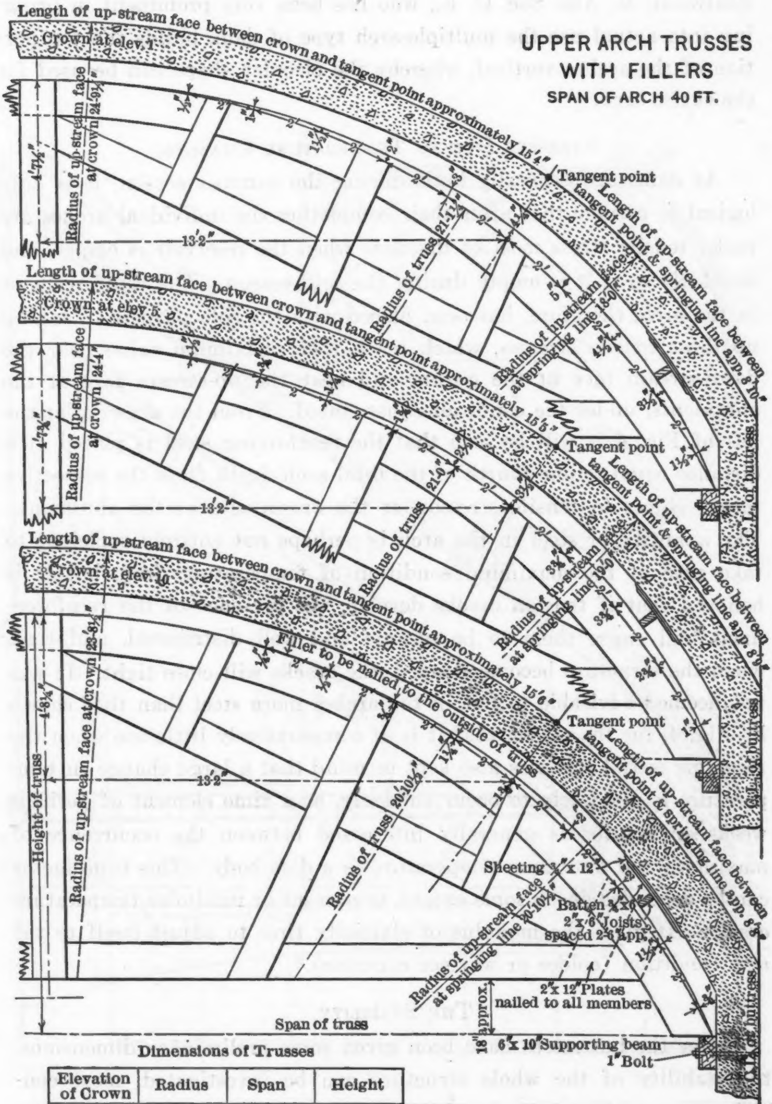
arch is divided into twenty voussoirs, and the forces acting on these voussoirs are calculated. These forces, though not distributed uniformly, are, of course, symmetrical on both halves of the arch, with respect to the center line between two buttresses.

On the right half of each diagram are shown the weights of the different voussoirs resolved into their components. The weight of a voussoir is partly transmitted to the base through the lower portion of the arch, and partly supported by the arch. The last component is equal to the total weight of the voussoir multiplied by cos. 50°, and is set off vertically, acting through the center of gravity of each voussoir. This vertical load is resolved into one radial component and one perpendicular thereto. The radial component can now be added directly* to the water load on the same voussoir, which, of course, is also radial. The geometrical sum of the last named (perpendicular) components represents the increased axial stress toward the abutments.

On the left half of each diagram the total radial forces acting on the different voussoirs are indicated. A polygon of forces is drawn, and, by using this, the line of pressure of the arch can be determined by drawing lines parallel to the proper rays in the polygon of forces. It is plainly seen that the line of pressure lies decidedly outside the center line of the arch; especially is this true of the arch with its crown at Elevation 5 (Fig. 1).

Using these diagrams, the correct shape of the wooden arch trusses supporting the form work for these upper elevations can be ascertained. The outside members of these trusses are made elliptical above Elevation 15, as shown in detail on Fig. 3, in such a way as to bring about coincidence between the line of pressure and the center line of the arch. At Elevation 15 the center line of the arch and the line of pressure do not exactly coincide, but they are close enough together to allow the circular shape of the arch to be used with entire safety. Below Elevation 15 the arch is made circular in a plane perpendicular to its sloping axis, and above Elevation 15 it is made elliptical in accordance with the diagrams, Figs. 1 and 2, or a slight modification thereof, depending on how far below the crest the maximum water level is to be. It might not be out of place to state here that J. S.

* For greater accuracy, this force should be multiplied by the ratio, $\frac{\text{length of mean radius}}{\text{length of up-stream radius}}$. It has been neglected in the present case, as this ratio is very close to unity.




**Dimensions of Trusses**

| Elevation of Crown | Radius | Span | Height |
|---|---|---|---|
| 1 | 21' 1¼" | 37' 8" | 11' 7½" |
| 5 | 21' 0" | 37' 6½" | 11' 7¾" |
| 10 | 20' 10½" | 37' 4½" | 11' 7¾" |

FIG. 3.

Eastwood, M. Am. Soc. C. E., who has been very prominent in bringing into actual use the multiple-arch type of dam, builds the top portion of the arches vertical, whereby the circular shape can be used for the entire arch.

### Stresses Due to Temperature Changes.

As dams are generally built during the summer season, it is only logical to assume that after their completion the individual arches are under tensile stress most of the time when the reservoir is empty, and decidedly so if it is empty during the cold season. The reinforcement in the arch, therefore, has been placed with the sole purpose of taking up these tensile stresses, which reach their maximum value near the down-stream face at the crown, and near the up-stream face at the abutments, under the conditions just stated. From the drawing at the left of Fig. 4 it will be seen that the reinforcing steel is placed at a distance equal to one-fourth of the total arch depth from the respective faces subject to tensile stresses, at the crown and at the abutments. The quantity of steel in the arch is perhaps not entirely sufficient to take care of the maximum condition of temperature drop, but it is believed that if tension cracks develop, the presence of the reinforcement will cause them to be minute and well distributed, and that, when the structure becomes loaded, the cracks will close tight. It was not deemed advisable to put in the arches more steel than that shown by Fig. 4, for the reason that it is of comparatively little use when the reservoir is full. It was also kept in mind that a large change in temperature is not likely to occur suddenly, as a time element of perhaps weeks or months is generally interposed between the occurrence of maximum and minimum temperature in a dam body. This time factor can be depended on, to some extent, to prevent or minimize temperature cracks. It gives the modulus of elasticity time to adjust itself to the new condition (colder or warmer concrete).*

### The Stability.

After the buttresses have been given some preliminary dimensions, the stability of the whole structure can be investigated, most con-

* That the deformation of concrete increases, under sustained load, has been shown by tests and in practice. Two papers on the subject were read at the Twelfth Annual Convention of the American Concrete Institute, in Chicago, one by A. H. Fuller, M. Am. Soc. C. E., and C. C. More, Assoc. M. Am. Soc. C. E., entitled "Tests Showing Continued Deformation under Constant Load", and one by Mr. Carl B. Smith entitled, "The Flow of Concrete Under Restrained Load". This was abstracted in the *Engineering Record,* March 4th, 1916, p. 329. See also "The University of Minnesota Studies in Engineering, No. 3", by F. R. McMillan.

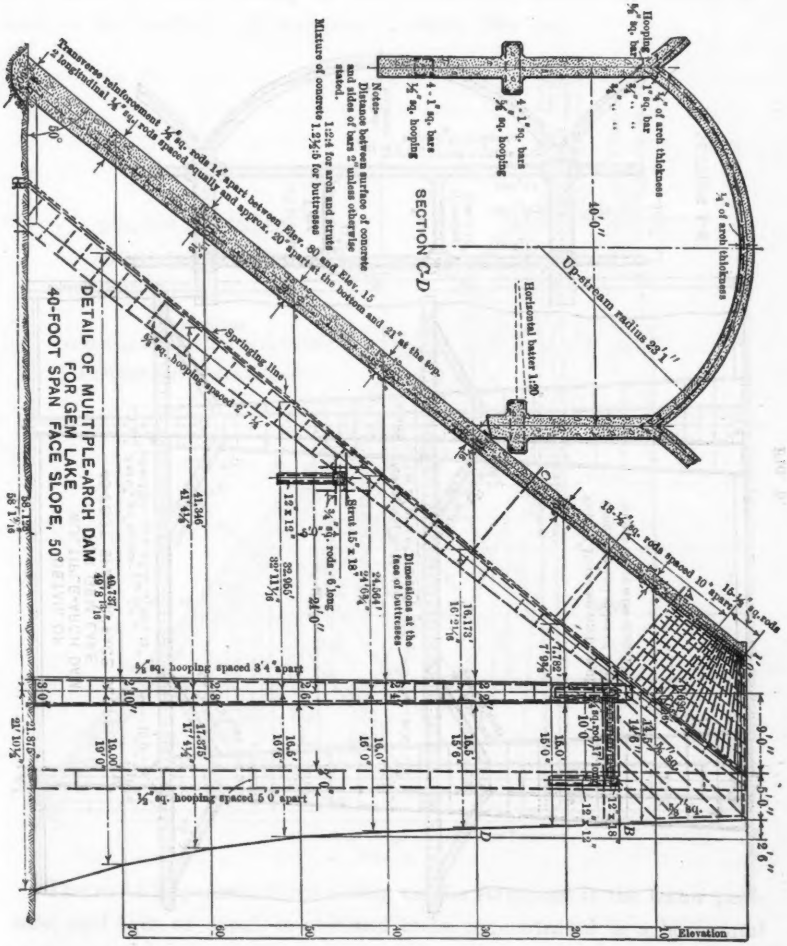

DETAIL OF MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN FACE SLOPE, 50°
SECTION C-D
Up-stream radius 23'1"
Horizontal batter 1:20
Transverse reinforcement ½" sq. rods 14" apart between Elev. 80 and Elev. 15
2 longitudinal ½" sq. rods spaced equally and approx. 20" apart at the bottom and 21" at the top.
Springing line
⅝" sq. hooping spaced 2' 7½"
⅝" sq. hooping spaced 3' 4" apart
½" sq. hooping spaced 5' 0" apart
Strut 15" x 18"
12 x 12
¾" sq. rods - 6' long
Dimensions at the face of buttresses
18-½" sq. rods spaced 10" apart
15-½" sq. rods
Note-
Distance between surface of concrete and sides of bars 2" unless otherwise stated.
Mixture of concrete 1:2½:5 for buttresses
1:2:4 for arch and struts
4-1" sq. bars
½" sq. hooping
⅝" sq. hooping
Hooping
⅝" sq. bar
1" sq. bar
¼" of arch thickness
40'-0"
Elevation
50°

FIG. 4.

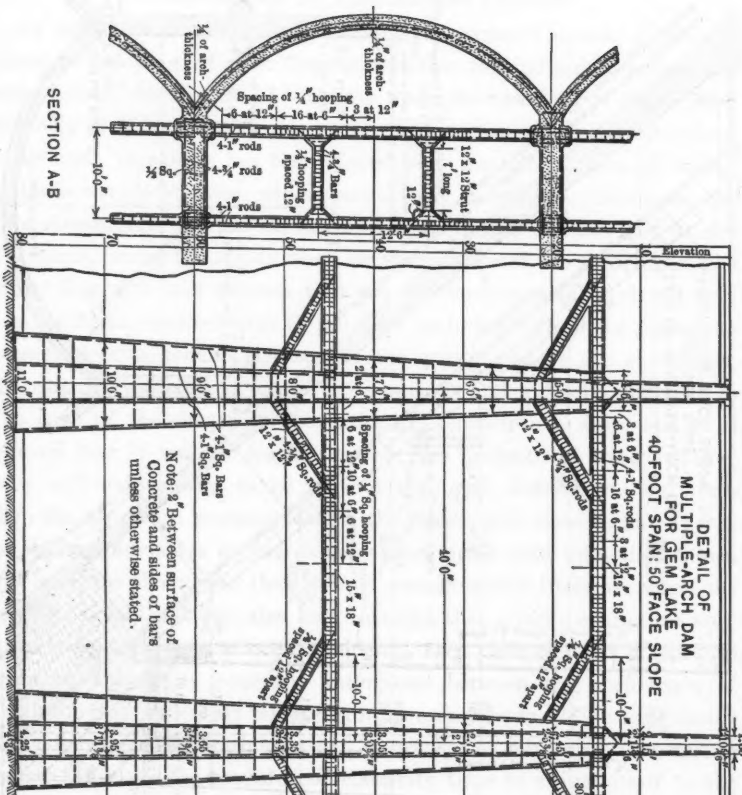

DETAIL OF
MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN: 50° FACE SLOPE
SECTION A-B
Elevation
Note: 2" Between surface of Concrete and sides of bars unless otherwise stated.
Spacing of ¼" hooping
4-1" rods
4-¾" rods
12" x 12" Strut 7' long
40'0"
10'0"

Fig. 5.

veniently by a graphical method, as shown by Fig. 6, representing a section through the crown of the arch. To facilitate the investigation, the dam is divided into horizontal sections, 10 ft. apart in elevation, and the forces acting on and above each section are shown to scale in the location and direction in which they act.

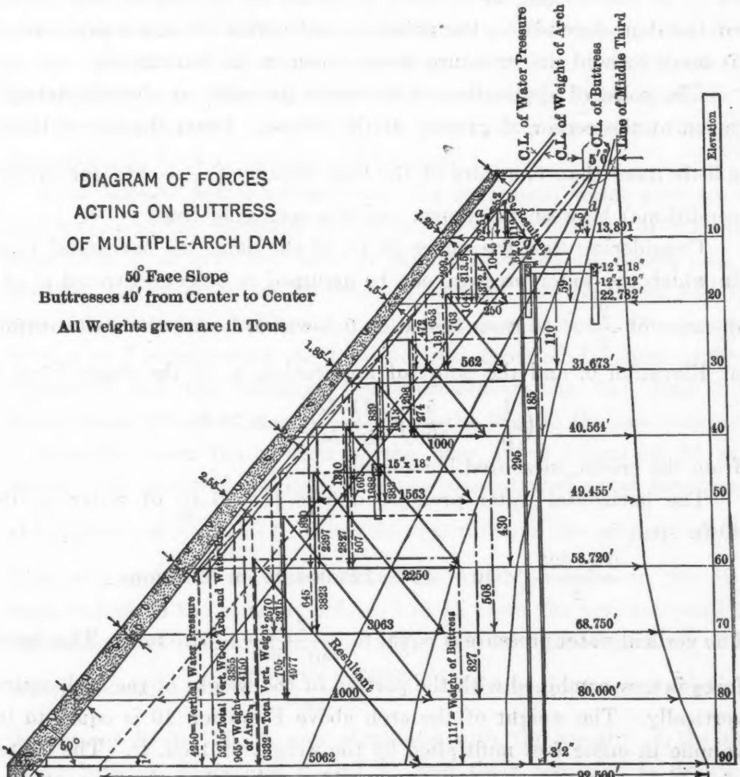



Fig. 6.

The most important force acting on the structure is the water pressure, and this, as usual, is assumed to be concentrated in a horizontal plane which is two-thirds of the total depth below the water surface.

The horizontal plane in which the water pressure is assumed to be concentrated intersects the up-stream face along an elliptical curve. The point of application of a single force, representing the water pressure on one total span of 40 ft., coincides with the center of gravity of

this ellipse, at least, as long as the water pressure does not penetrate the up-stream face skin, such as plastering, etc.

Due to the fact that the arches have been given a slope of 50° with the horizontal, the water pressure will have a vertical component $= \dfrac{\text{horizontal component}}{\tan. 50°}$. To a very great extent, the stability of the dam depends on the presence and action of this component, as it tends to hold the structure firmly down on its foundation.

The point of application of the water pressure, as already stated, is taken at the center of gravity of the ellipse. From the crown this is a little more than one-third of the total distance $\left(\frac{1}{3} + 5\%\right.$ for present conditions) between the crown and the springing line.

Considering first the upper 10 ft. of the dam, the horizontal plane in which the water pressure can be assumed to be concentrated is at a distance of $\frac{2}{3} \times 10$ from Elevation 0 downward, with the water surface at Elevation 0, and the point of application, *a*, of the single force is

$$\left(\frac{1}{3} + \frac{5}{100}\right) \frac{11.55}{\sin. 50°} = 5.78 \text{ ft.}$$

from the crown, measured horizontally.

The horizontal water pressure due to the 10 ft. of water on the 40-ft. span is

$$\frac{0 + 625}{2} \times 10 \times 40 = 125\ 000 \text{ lb., or } 62.5 \text{ tons.}$$

The vertical water pressure is equal to $\dfrac{62.5}{\tan.\ 50°} = 52.5$ tons. This latter force is now combined with the portion of the weight of the arch acting vertically. The weight of the arch above Elevation 10 is equal to its volume in cubic feet multiplied by the weight of 1 cu. ft. The center of gravity of the section—a trapezoid—is found, and the center of gravity, *b*, of the whole arch is then taken to coincide with the center of gravity of an ellipse through the center of gravity of the section, in the same manner as explained previously for the water pressure. The weight of the arch is now combined with the vertical component of the water pressure, and the location of their resultant is found by taking moments around either point, *a* or *b*. The numerical value of the resultant is equal to the sum of the two forces, 52 + 52.5 = 104.5 tons. Taking moments around the point, *a*, and scaling the distances,

preferably on the sloping line between $a$ and $b$ for greater accuracy, we have the equation $\frac{52 \times 2.75}{104.5} = x = 1.367$ ft., giving the location of the resultant at a point 1.367 ft. from $a$ along the line between $a$ and $b$. The weight of the buttresses (above Elevation 10) assumed to be concentrated in the center of gravity, $d$, is calculated to be 14.5 tons; it is combined with the vertical load of 104.5 tons on the arch, in the same manner as shown previously, by taking moments around any point, say $c$. The location of the resultant is found to be at $e$, and its value is $104.5 + 14.5 = 119$ tons. This represents the total vertical force, both as to size and location.

This force is now combined with the horizontal water pressure of 62.5 tons, and the resultant is drawn. It is the purpose of this diagram, Fig. 6, to give the value of the resultant of all forces, and to establish the point of intersection, $g$, between this resultant and the base, in this case at Elevation 10. For convenience, the distance from $e$ to $f$ representing the total vertical force of 119 tons, may be measured, and the distance, $f$-$g$, representing the 62.5 tons horizontal water pressure, may be set off on the base to the same scale.

Consider next the portion of the dam above Elevation 20 as a whole. The place of application of the concentrated water pressure is at a distance of $\frac{2}{3} \times 20$ ft. below Elevation 0, and the point of application of a single force representing the water pressure on the 40-ft. span is located the same as before, 5.78 ft. from the crown toward the springing line, measured horizontally. The vertical component of the water pressure $= \frac{\text{horizontal component}}{\tan. 50^\circ}$, acting on the arch between Elevations 0 and 20, is now combined with the weight of the arch lying between Elevations 0 and 20, and their resultant is drawn in the correct location, found by taking moments as shown previously. This resultant is again combined with the weight of the portion of the buttress lying between Elevations 0 and 20. The shape of the buttress is taken as that of an obelisk, and its volume and the location of the center of gravity are found from ordinary rules applying to such bodies. The weight of the struts and counterforts is to be added to the weight of the buttress, whereby the location of the center of gravity might be slightly changed.

The resultant of all vertical forces acting on the dam above Elevation 20 can now be found, both as to size and location, and combined with the horizontal water pressure, whereby the point of intersection of the resultant with the base at Elevation 20 is determined.

The same method of procedure is followed for the remaining portion of the dam, each time taking the base 10 ft. lower than in the preceding calculations, and the whole diagram, Fig. 6, is completed. If, now, the center line of the buttress, and also the two lines representing the middle thirds, are drawn, this diagram will point out very clearly whether or not the load is distributed economically on the buttress. For maximum economy, the resultant of all forces should intersect the base in the center line of the buttress; then the load will be distributed uniformly over the whole base. Toward the top this is not quite possible, and is not important, as the material there cannot be stressed very highly at any rate; but, toward lower elevations, the down-stream slope of the buttress should be shaped so as to conform with this condition, *viz.*, the resultant intersecting the base in the center line, or approximately in the center line, of the buttress. The total vertical load on the section shown in Fig. 6 is seen to be 6 332 tons, and the horizontal water pressure, 5 062 tons, both on a 40-ft. span. If the coefficient of friction is taken at 0.75, it is seen that the actual shear along the base amounts to only $5\,062 - 6\,332 \times 0.75 = 313$ tons. There is considerable steel in the section to help take up this shear, and therefore it was not deemed necessary to eliminate the shear entirely. There is no hydrostatic uplift to amount to anything acting on a dam of this type, and water could hardly find its way to lubricate the surfaces of possible cracks in the buttresses. Wherever it would be desirable to eliminate the shear entirely, the face slope should be made flatter, say 45, instead of 50 degrees. This, of course, adds to the material required for construction, but is the cheapest and best way of accomplishing the result.

Some difference of opinion may well exist as to the actual stress per square unit of area of the buttress at any horizontal elevation. The loads per buttress are given on Fig. 6. For instance, at Elevation 80, the vertical load is seen to be 4 977 tons, and the horizontal load, 4 000 tons. As the horizontal area is 360 sq. ft., the unit vertical stress should be $\frac{4\,977}{360} = 13.82$ tons per sq. ft., or 192 lb. per sq. in. The

shear would be a little more than friction alone would take care of, but steel is provided for the remainder. The resultant of the two forces (horizontal and vertical) intersects the base 2 ft. down stream relative to the center line, but on account of having the counterfort (20 sq. ft.) on this side of the center line also, the stress is actually distributed uniformly. Now, if, instead of the two principal forces, their resultant is used, assuming it as acting on a number of steps perpendicular to the direction of this force (the resultant), the apparent unit stress will be much higher. Thus, the resultant (6 385 tons) acting on an area equal to the sum of all the steps ($360 \times \sin. 51° 13' = 280$ sq. ft.) will produce a compression equal to $\frac{6\,385}{280} = 22.8$ tons per sq. ft., or 317 lb. per sq. in.

There is a great difference between 192 and 317 lb. per sq. in. The actual unit compression will be somewhere between, undoubtedly depending on the relative value of the modulus of elasticity of the concrete for compression and for shear. It is seen that the first method, using the principal forces, gives the maximum possible shear that could occur, and that the other method gives the maximum possible unit compression that could occur. None of these methods is very satisfactory, as none fixes the absolute value of stress within narrow enough limits, but the writer knows of no better at present.

The reinforcing steel embedded in the buttress is put there for different purposes. Along the up-stream slope, the triangular steel construction shown in Figs. 4 and 5 ties the adjacent arches into the buttress. This is desirable on account of the fact that, in order to facilitate construction, the buttresses are built first, and the arches later. The hooping that interconnects the different bars is simply left protruding through the concrete of the buttress at the time this is built. Should one arch fail, this triangular girder would immediately take up the unbalanced thrust and prevent adjacent arches from collapsing, and this is its principal duty. Besides this, however, the steel is active in preventing cracks in the buttress and in taking up some shear. Toward the down-stream edge of the buttress, four vertical rods are embedded in the concrete for the purpose of stiffening it, preventing cracks, and taking care of wind pressure. Toward the middle portion, reinforced counterforts are constructed for the same purpose. These are still more effective in accomplishing this, due to

the greater distance between the pairs of steel rods. All material in the counterforts supports load the same as the buttress itself, but the counterforts, at the same time, are most effective in stiffening the large flat slab (buttress) so that fewer struts are required.

Toward the top of the buttress (at Elevation 15) two horizontal struts are tied to the vertical reinforcement (Figs. 4 and 5). These two struts are designed so that, besides their main purpose of holding the upper portion of the buttress in place, they are capable of supporting a light roadway. At Elevation 45 another strut is placed near the up-stream face, mainly in order to support the triangular girder, should the latter ever be required to support any unbalanced arch pressure. All struts are able to withstand tension as well as compression, as can be judged from an inspection of the details on Figs. 4 and 5.

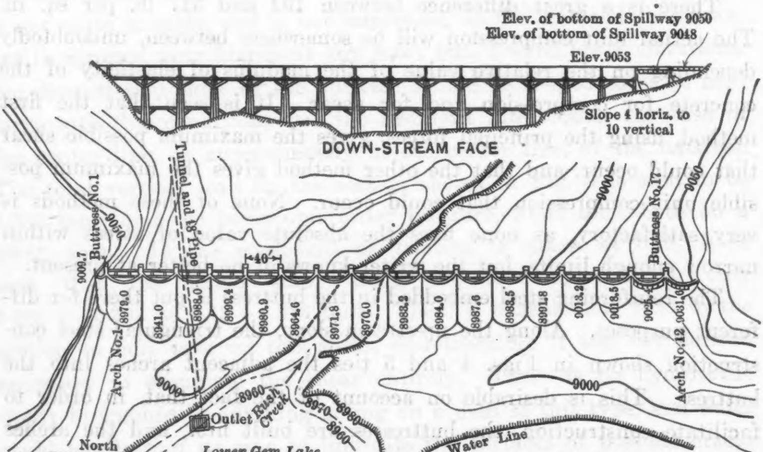



FIG. 7.

## GEM AND AGNEW LAKE DAMS.

The design shown in the foregoing illustrations was made by the writer for the Pacific Power Corporation, of Bodie, Cal., to be used for the construction of their Gem and Agnew Lake Dams, on Rush Creek, Mono County, California. Fig. 7 is a plan of the Gem Lake Dam and dam site, and Fig. 8 shows the Agnew Lake dam site. On Fig. 7 is also indicated the outlet works, consisting of a concrete

chamber provided with a row of iron bars in front, and a 48-in. steel pipe laid through a tunnel for about 300 ft. This pipe line terminates in a power-house 1 808 ft. below (in elevation) the top contour of the lake. The maximum height of any individual arch of this dam is 84 ft., and the vertical distance from the deepest point in the foundation to the crest is 112 ft. The length across the dam site at the crest (Elevation 9 053) is 700 ft. The reservoir created has a capacity of 17 000 acre-ft., and is capable of regulating Rush Creek to yield an estimated average flow of 45 sec-ft. The drainage area above the dam is 22½ sq. miles, and is on the eastern slope of the Sierra Nevada Mountains. Its elevation ranges from 9 000 ft.

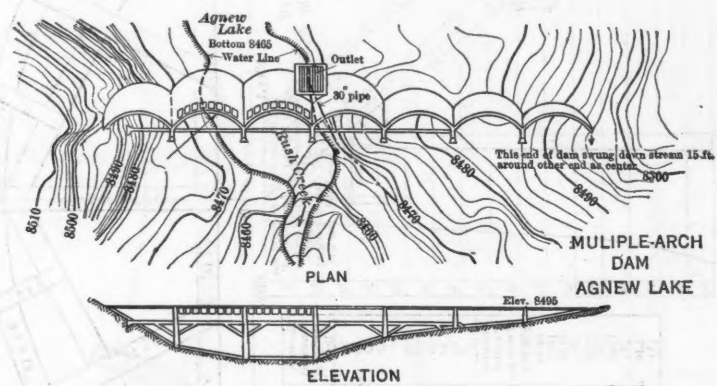
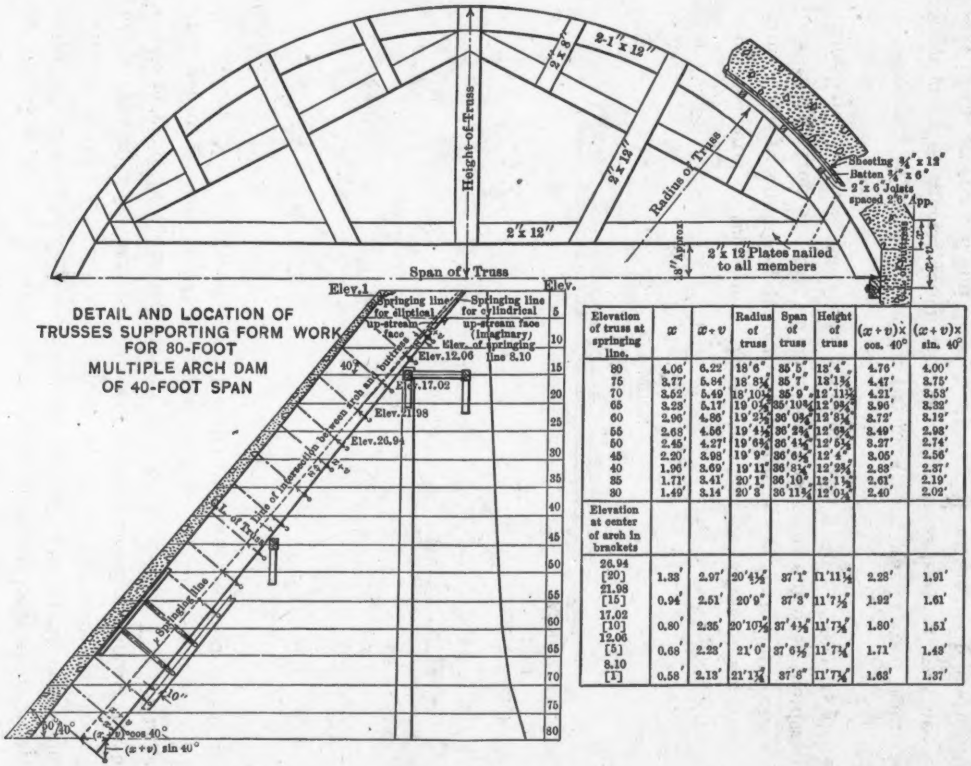



FIG. 8.

above sea level at the dam to 12 000 ft. along the crest of the mountains. The precipitation at these elevations is still high, but decreases very rapidly from the foot of the steep mountains out over the desert immediately east, at elevations ranging from 6 000 to 7 000 ft. above sea level.

The rock at the dam site, for the most part, was exposed bed-rock worn clean by glacial action. Some excavation, however, was necessary in the stream bed and through a rock slide at the north abutment. The building material for the dam was found near-by. The sand was taken from the shore of the natural lake. The rock had to be hauled a short distance on a tramway, first from the outlet tunnel dump (limestone), and later from a large rock slide (granite) about




| Elevation of truss at springing line. | $x$ | $x+v$ | Radius of truss | Span of truss | Height of truss | $(x+v)\times$ cos. 40° | $(x+v)\times$ sin. 40° |
|---|---|---|---|---|---|---|---|
| 80 | 4.06' | 6.22' | 18'6" | 35'5" | 13'4" | 4.76' | 4.00' |
| 75 | 3.77' | 5.84' | 18'8¼" | 35'7" | 13'1¾" | 4.47' | 3.75' |
| 70 | 3.52' | 5.49' | 18'10¼" | 35'9" | 12'11½" | 4.21' | 3.53' |
| 65 | 3.23' | 5.17' | 19'0½" | 35'10¾" | 12'9¾" | 3.96' | 3.32' |
| 60 | 2.96' | 4.86' | 19'2½" | 36'0¾" | 12'8¼" | 3.72' | 3.12' |
| 55 | 2.68' | 4.56' | 19'4½" | 36'2¾" | 12'6¾" | 3.49' | 2.93' |
| 50 | 2.45' | 4.27' | 19'6¾" | 36'4½" | 12'5½" | 3.27' | 2.74' |
| 45 | 2.20' | 3.98' | 19'9" | 36'6½" | 12'4" | 3.05' | 2.56' |
| 40 | 1.96' | 3.69' | 19'11" | 36'8¼" | 12'2¾" | 2.83' | 2.37' |
| 35 | 1.71' | 3.41' | 20'1" | 36'10" | 12'1½" | 2.61' | 2.19' |
| 30 | 1.49' | 3.14' | 20'3" | 36'11¾" | 12'0¼" | 2.40' | 2.02' |
| Elevation at center of arch in brackets | | | | | | | |
| 26.94 [20] | 1.33' | 2.97' | 20'4½" | 37'1" | 11'11½" | 2.28' | 1.91' |
| 21.98 [15] | 0.94' | 2.51' | 20'9" | 37'3" | 11'7½" | 1.92' | 1.61' |
| 17.02 [10] | 0.80' | 2.35' | 20'10½" | 37'4½" | 11'7¼" | 1.80' | 1.51' |
| 12.06 [5] | 0.68' | 2.23' | 21'0" | 37'6½" | 11'7¼" | 1.71' | 1.43' |
| 8.10 [1] | 0.58' | 2.13' | 21'1¼" | 37'8" | 11'7¼" | 1.63' | 1.37' |

FIG. 9.

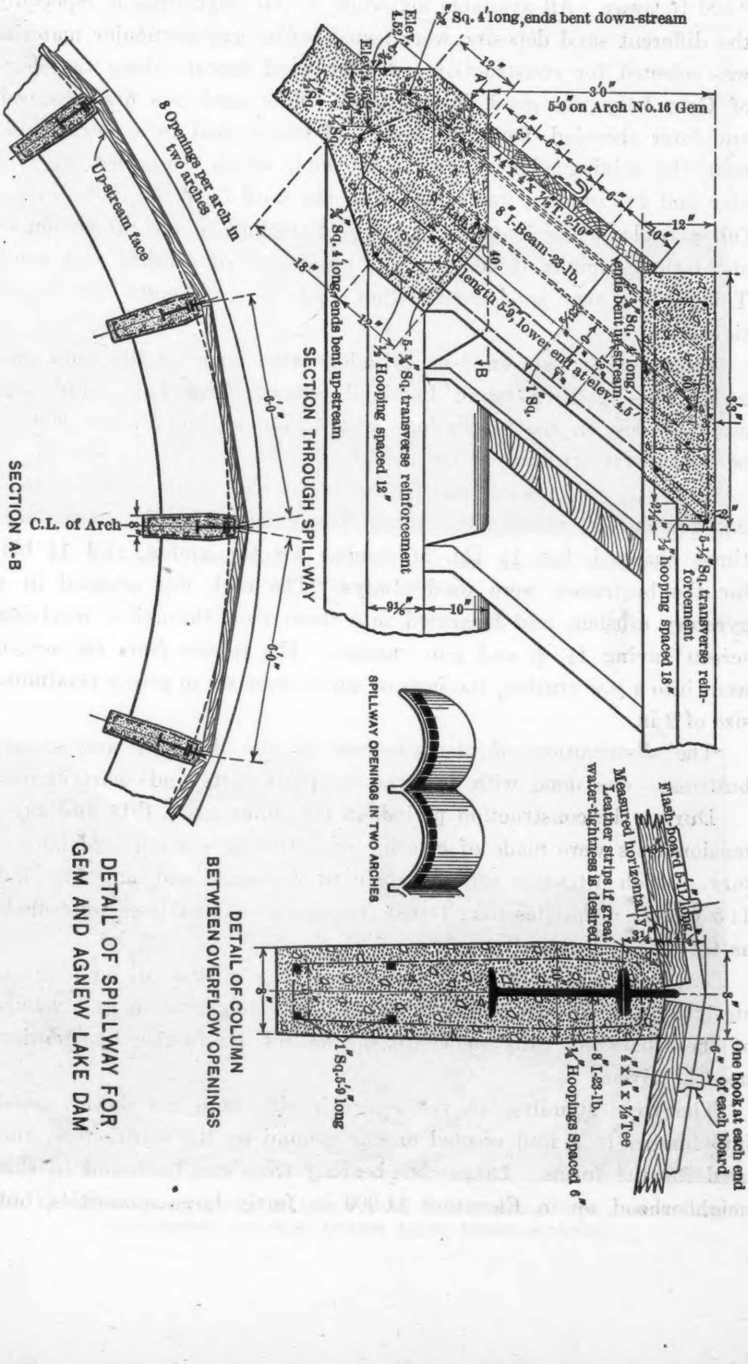



FIG. 10.

DETAIL OF SPILLWAY FOR GEM AND AGNEW LAKE DAM

2 500 ft. away. All available materials in the neighborhood, especially the different sand deposits, were tested before any particular material was selected for construction. As the sand deposit along the shore of Gem Lake was good, it was used. This sand was first pumped, and later shoveled, from the lake, and transported to a storage pile near the mixing plant. This lake sand, which contained 3½% of clay and 1% of dirt, was mixed with the sand from the rock crusher (all particles being less than ¼ in. in diameter) in the proportion of about three-fourths of lake sand to one-fourth of crushed rock sand. This gave a very good combination, both as to strength and water-tightness.

Compression tests on 6-in. cylinders were made as the work progressed, using Bear brand Portland cement, Gem Lake sand, and crushed rock, in the proportions 1:2:4, and averaged about 900 lb. per sq. in. for crushing at the age of 14 days.

A 1:2:4 mix was adopted for the arches and struts, and a 1:2½:5 mix for the buttresses. The actual proportions, however, were sometimes changed, but 1½ bbl. of cement for the arches, and 1¼ bbl. for the buttresses were used always. The rock was crushed in a gyratory crusher, and separated into three sizes through a revolving screen having 1½, ¾, and ¼-in. meshes. The rejects from the screen went into a jaw crusher, the jaws of which were set to give a maximum size of 2 in.

The distribution of the concrete to the different arches and buttresses was done with two-wheeled push carts and short chutes.

During the construction period, in the summers of 1915 and 1916, tension tests were made of cement briquettes in a small field laboratory. Much attention was also paid to the sand used, and 1:2 and 1:3 mortar briquettes were tested frequently; silt analyses were made as the work progressed.

The reinforcement placed in the dam consisted of high-carbon steel bars, either corrugated or twisted. As the position and details of these bars are fully shown in the figures, no further explanation need be given.

The trees standing on the reservoir site were cut down, sawed into lumber in a mill erected on the ground by the contractors, and used for the forms. Large cone-bearing trees can be found in this neighborhood up to Elevation 11 000 in fairly large quantities, but

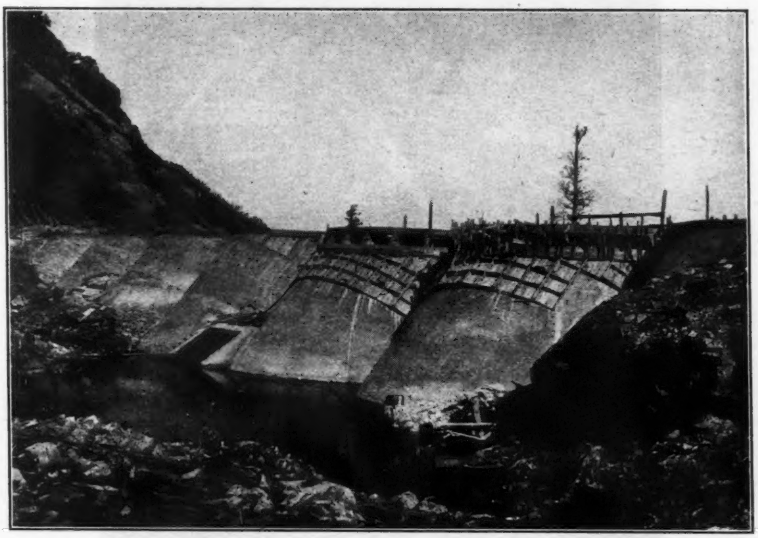


FIG. 11.—UP-STREAM FACE OF AGNEW LAKE DAM, SHOWING SPILLWAY OPENINGS AND INLET.

FIG. 12.—DOWN-STREAM FACE OF AGNEW LAKE DAM, SHOWING SINGLE STRUT AND BUTTRESSES, AND ALSO COPING ALONG CREST OF ARCH.

FIG. 13.—GEM LAKE DAM.

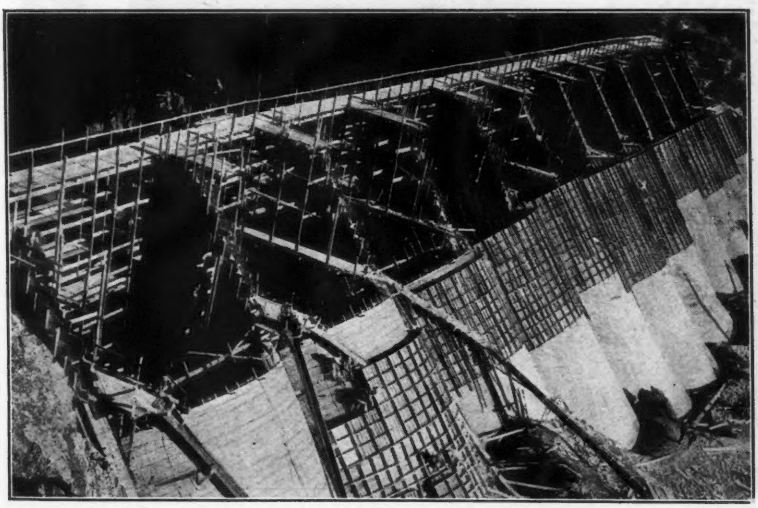


FIG. 14.—GEM LAKE DAM, SHOWING SECTION OF UP-STREAM FORM BEING HOISTED INTO PLACE, ALSO MAIN UPPER RUNWAY WITH BRANCHES ALONG BUTTRESSES.

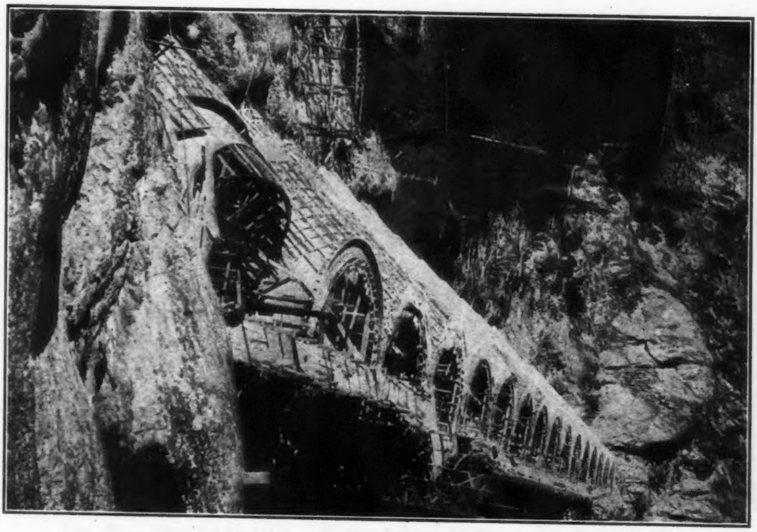


FIG. 15.—GEM LAKE DAM, SHOWING PROGRESS OF FORM BUILDING.

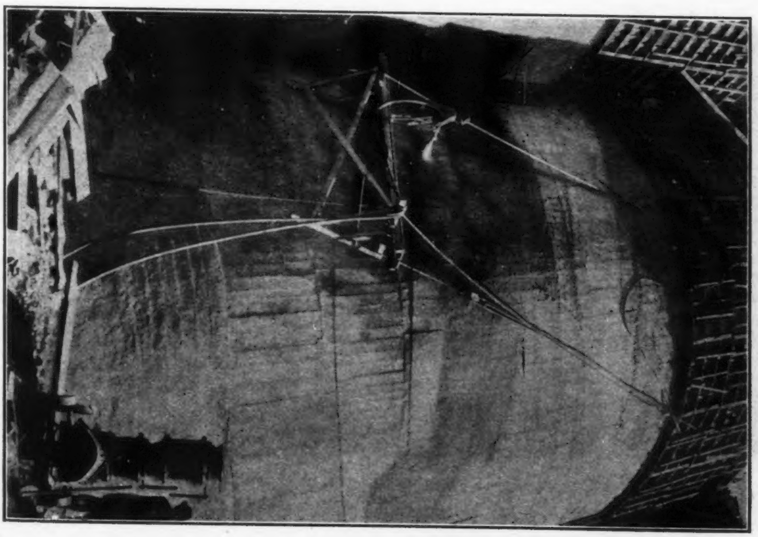


FIG. 16.—APPLYING PLASTER TO UP-STREAM FACE WITH CEMENT GUN.

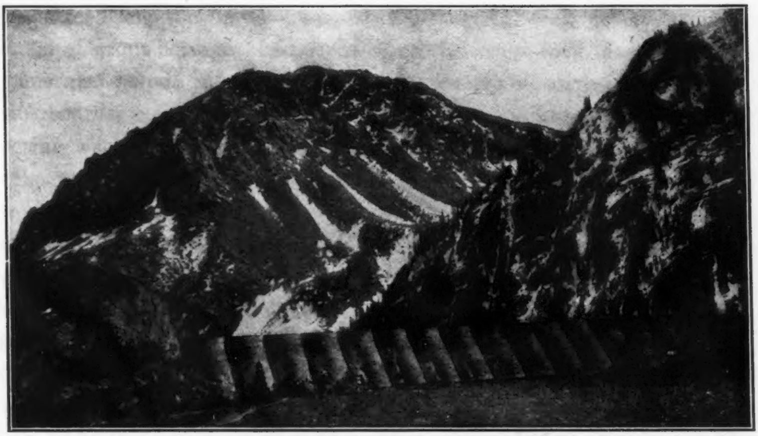


FIG. 17.—GEM LAKE DAM. WATER LEVEL 35 FT. ABOVE TUNNEL FLOOR.

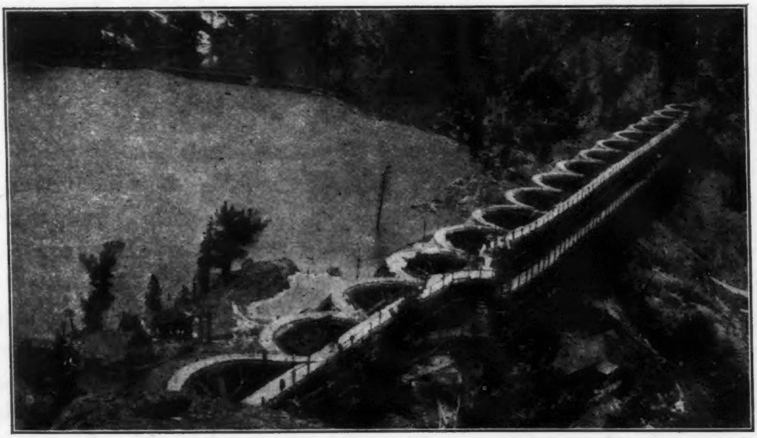


FIG. 18.—GEM LAKE DAM, SHOWING RUNWAYS AND CONCRETE ELEVATOR.

not of the best quality. The lumber proved to be good enough for form work, was used green, and on that account $\frac{3}{4}$ by 4-in. battens were placed on the under side of the sloping arch to avoid the leakage of grout between the boards when shrinkage took place. These battens also helped to give an even, circular shape to the outer $\frac{3}{4}$ by 12-in. boards, constituting the form for the down-stream face. This sheeting was supported on 2 by 6-in. studs, 2 ft. 6 in. apart, toe-nailed to the wooden arch trusses shown in Figs. 3 and 9. These trusses were 5 ft. apart in elevation, and a little more than $6\frac{1}{2}$ ft. apart, measured along the arch slope. The form work for the up-stream face consisted also of $\frac{3}{4}$ by 12-in. boards nailed to 2 by 6-in. studs, which in turn were nailed to circular boards at regular intervals. This outside form was held away from the inside form (down-stream side) by wooden distance pieces of the proper length, which pieces were removed just before the concrete reached them.

A 1 : 2 plaster coat of cement mortar $\frac{1}{4}$ in. thick at the crest, and increasing to $\frac{3}{4}$ in. thick 80 ft. below, was put on the up-stream face with a cement gun.

At the south abutment the two last arches are provided with spillway openings. These spillway openings can be closed with loose flash-boards. This it is proposed to do toward the end of the wet season, so as to fill the reservoir to within, say 1 ft. of the crest of the dam, thereby gaining 2 ft. of water over an area of nearly 300 acres. The spillway is shown in detail on Fig. 10.

## Cost.

The Gem Lake Dam contains 8 537 cu. yd. of concrete and 82 tons of reinforcing steel. The contract price was $22 per cu. yd., including cement, forms, plastering the up-stream face, and all tools and materials except the reinforcing steel, which was paid for as an extra at the rate of $110 per ton in place. The excavation, of which there was only a limited quantity, was also paid for as an extra. The high cost is explained by the fact that freight rates were high, and, further, that the distance from the railroad to the power-house site at Silver Lake, at the foot of the steep mountains, was nearly 60 miles over desert roads with heavy grades. From the power-house site, a tramway, approximately 4 500 ft. long, took all supplies up the mountain side to Agnew Lake, 1 250 ft. above. At the outlet of this lake, the

Agnew Dam, a smaller structure, 30 ft. high and 280 ft. long, similar in design to the Gem Lake Dam, but having only one strut, was built at the same time and at the same unit prices, in order to increase the capacity of the natural lake. All material for use in the construction of the Gem Lake Dam was brought across this lake on a barge, for a distance of about 2 000 ft., to the foot of another tramway terminating at the site of the Gem Lake Dam, 550 ft. higher in elevation. The cost of these tramways was not included in the price per cubic yard for concrete, but was paid for as an extra. These tramways were also necessary for the construction of the pressure pipes to the power-house, one from Gem Lake, and one from Agnew Lake, and would have had to be built independent of the dams.

For the long haul across the desert from the Southern Pacific Railroad station at Benton, Cal., to the power-house at Silver Lake, six 75-h.p. C. L. Best tracklayers, burning distillate, were used, each hauling three trailers. The net load was as close to 20 tons as practicable, and the time necessary to make one round trip was about 6 days of 12 hours, including loading, unloading, and ordinary delays. The speed per hour of these tracklayers was 2½ miles on high gear, and 1½ miles on low gear. The contractors found that the cost to them for hauling this distance was at the rate of $13.50 per ton. The hauling of the cement and materials for the dams was not paid for extra, but included in the price for concrete in place.

A rock fill dam on the same site would have had to be built for $2.15 per cu. yd. (construction cost), including the water-tight face and hand-laid rock, in order to be on equal terms, as to cost, with the multiple-arch dam built. In the writer's opinion, this would not have been possible in this place. In any locality where cement can be laid down at less cost than in this case, the relative cost of a multiple-arch dam and a rock fill dam will be still more in favor of the multiple-arch dam.

The writer furnished the designs and also supervised the construction of the dams.

Mr. C. O. Poole was Chief Engineer for the whole development, E. J. Waugh, Assoc. M. Am. Soc. C. E., was Resident Engineer, L. B. Curtis, M. Am. Soc. C. E., Field Engineer, and Mr. F. O. Dolson, Superintendent of Construction.

Messrs. Duncanson Harrelson Company, of San Francisco, were the contractors for both dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## CEMENT JOINTS FOR CAST-IRON WATER MAINS

By Clark H. Shaw, Assoc. M. Am. Soc. C. E.

To be Presented April 18th, 1917.

### Synopsis.

This paper presents the method of making successful cement joints in cast-iron water mains.

A brief history of the use of this joint establishes the fact that this method of construction has long since passed the stage of experiment, and has been proved to be an economic factor in laying such mains.

The process of making the joints is described and illustrated, some experiments in jointing are described, cases showing the strength of cement joints under trying conditions are cited, and data relating to cost, etc., are presented.

About 1886 a cast-iron pipe line for water distribution was laid with cement joints at Redlands, Cal., and in 1891 joints of that kind were used at Los Angeles, Cal., but evidently with questionable results, as the method was not adopted. In January, 1907, Mr. Charles Thornburg, then Superintendent of one of the water companies operating in Long Beach, Cal., decided to try cement joints for a 16-in.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

cast-iron pumping main, and instructed his foreman of construction, Mr. F. M. Shrode, to conduct some experiments.

No definite process was outlined to the foreman, but his experiments and practice in repairing steel riveted water mains under pressure, by using a dry mixture of neat cement in caulking the bands around these pipes, gave him an idea that a moist cement could be caulked into the bell solidly, and would produce the results desired. It was probably Mr. Shrode, who, by this experiment, finally perfected the joint and used it in the construction of the entire line. When this line was completed and put into service, working under a static head of about 190 ft., several places showed some seepage, particularly at the lower end of the line, where the work was started, and it was decided to re-caulk these joints at the first opportunity; it was noticed, however, that the moisture was gradually drying up, and the seepage finally ceased.

Cast-iron construction was then abandoned by this company until 1911. During that year the works came into the possession of the municipality, and the writer was appointed Engineer of the Water Department. After looking into the merits of the cement joint, as used on this 16-in. pumping main, it was adopted as the proper method of construction, and since that time it has been used throughout the entire system.

Long Beach now has 60 miles of cast-iron water mains, ranging from 4 to 24 in. in diameter, laid with joints of this type. All these pipes are under pressures ranging from 40 to 80 lb. per sq. in., and are giving perfect satisfaction.

### Method of Making the Joint.

In making the cement joint the pipe is placed and spaced in the usual manner. A thin backing of the best dry jute is used instead of oakum, as the jute is free from oils and grease (which should be avoided). A Portland cement, conforming to the specifications advocated by the American Society for Testing Materials, is used. The dry cement is placed on a piece of canvas (usually a cement sack ripped open), and moistened just so that when thoroughly mixed by hand (Fig. 1) it will be of such a consistency that when gripped tight it will hold the form of the hand (Fig. 2), and when dropped 12 in. it will crumble (Fig. 3). The canvas containing the cement is placed

under the bell, and the cement is tamped into place by hand with a caulking iron until the bell is about half full (Fig. 7). It is then caulked with heavy blows until the cement is thoroughly packed in the back of the socket. This process is continued until the bell is packed solid out to the face (Fig. 8). A small bead of neat cement in a plastic condition is then put on, using the caulking iron as a trowel (Fig. 9). As soon as the initial set of the cement in the bead has taken place, the joint is covered with earth to protect it from the air and sun. In back-filling, the excavated material is always settled with water, which helps to cure the exposed portion of the joint.

The bead is essential, in the writer's opinion, as the cement packed in the bell is so dry that without protection it would absorb moisture from the water used in settling the trench, and it is believed that, should the joint develop seepage when the pressure is put on in the main, the cement, being dry, would expand and aid materially in keeping the joint tight.

Experiments on cement joints constructed without the bead showed that, 24 hours after completion, they absorbed water readily. In cases where seepage has developed and has subsequently closed, it is assumed that the dry cement absorbed the moisture from the inside, expanded, and filled the seepage pores.

About 20% of the cement is wasted by falling off the canvas or being thrown out by the caulker. If any dust or earth from the trench falls on the canvas or in the cement, it is immediately taken out, together with enough cement to make sure that the remainder is clean. In mixing the cement with water, care is taken that there shall be no lumps in the material, no matter how small. If any cement is left on the canvas when a joint is completed, it is used on the next joint, provided the work is continuous, otherwise new batches are made. Special blunt caulking tools are used (Fig. 4).

The joint is allowed to stand 48 hours before the pressure is turned on and the main is put into regular service. Cement joints have been used with satisfactory results, however, 12 hours after completion, but this is not considered safe practice. Pressure tests are never made by the writer prior to putting a main into service.

At San Diego, Cal., a pressure test was made by caulking a 6-in. cast-iron tee, one side of the tee being filled with a plug and each of the two ends filled with short lengths of cast-iron pipe with plugs caulked

in the ends. As the pieces of pipe caulked in the tee were scrap ends cut from other pipes, they had no bead on the joint end, and, notwithstanding the fact that the joint was made with smooth pipe, it took a pressure of more than 300 lb. per sq. in. to force the pipe out. The test was made about 48 hours after the joint was made.

In another test, made at Winnipeg, Man., three lengths of 6-in. pipe were laid with four cement joints, on January 13th, 1916. After 6 days, pressure was put on the pipe, in increments of 25 lb., and the joints were found to show no leakage or moisture, up to 125 lb. At 150 lb. one joint showed moisture on the surface of the cement.

On January 24th another test was made, and at one joint moisture appeared at 175 lb. On January 31st this joint showed moisture with 200 lb., and also on March 15th, with a pressure of 255 lb. This joint was the weakest of the four. The pressure was kept on the pipe about one-half hour in each case.

Fig. 5 is a section of a joint made with a cut piece of pipe, and shows the position of the jute and the cement when there is no caulking rim or bead on the end of the pipe (as in the San Diego test). The cement bead, as shown in Fig. 5, was made larger than usual, and covers the entire face of the bell, though normally it covers only about one-half.

The strength and rigidity of the cement joint are shown by the following instances where cast-iron mains have been subjected to severe tests: A new trenching machine was being tested by a sewer contractor in Long Beach, Cal. It operated parallel with and 5 ft. from a 6-in. cast-iron water main, with cement joints, which had been laid in January, 1915. The trench dug by this machine was 3 ft. wide and 18 ft. deep. Some time before noon on February 1st, 1915, the side of the trench next to the water main caved in, leaving about 40 ft. of the pipe hanging and supporting about 2½ ft. of earth on top of it. This condition was not reported until the next morning, and at nine o'clock braces were put in to support the pipe. The main was under a pressure of about 65 lb. per sq. in. at the time. There was not the slightest seepage from any of the joints as result of this strain.

Again, in June, 1915, 94 ft. of 4-in. cast-iron pipe with cement joints fell into a sewer trench, as the result of a cave-in under conditions similar to those just described. This pipe was under a pressure of about 55 lb. per sq. in. At one end it broke at a service connection;

FIG. 1.

FIG. 2.

FIG. 3.

FIG. 4.

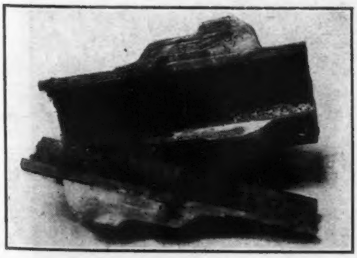


FIG. 5.

FIG. 6.

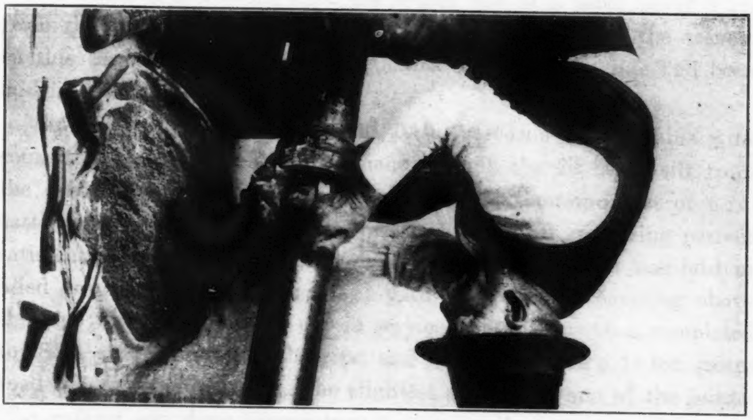


FIG. 7.

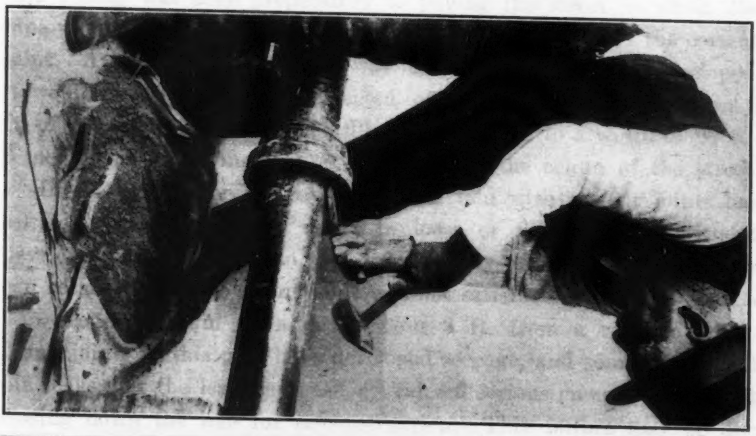


FIG. 8.

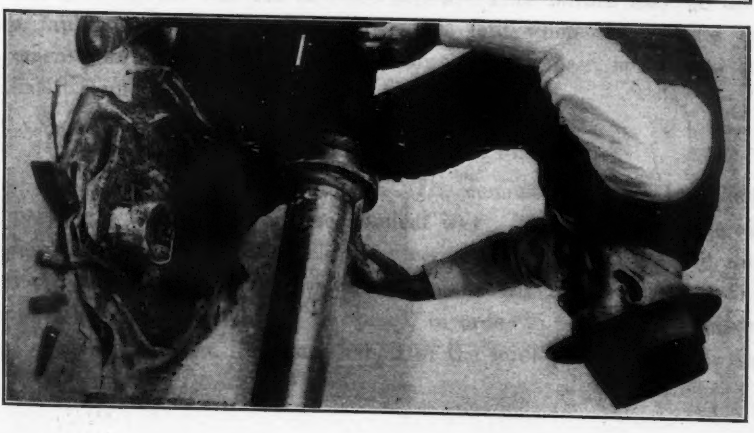


FIG. 9.

at the other end it broke near the spigot end of a pipe, about 8 in. from the joint, letting the whole 94-ft. section fall into the trench. In this case every cement joint remained intact. This line had been laid in March, 1913.

Several thousand feet of 8 and 10-in. cast-iron water mains with cement joints have been laid in made ground, the fill being silt from the dredging of harbor channels; also several thousand feet of 8-in. cast-iron pipe have been laid in fine beach sand; all are giving perfect satisfaction. An 8-in. cast-iron main with cement joints was laid in filled ground (the soil being clay) with only 6 in. of covering above the pipe when the line was put in service; the fill was then completed to 18 in. above the top of the pipe, and was rolled with a 14-ton steam road roller, without causing the slightest seepage in any of the joints. The rolling was done preparatory to paving the street in which the pipe was laid. This was a very severe test of the merits of the cement joint. At the time this rolling was done, had any seepage developed, it would have been readily detected, as the whole line was within 3 ft. of the edge of the fill. The fill was completed to a width of 80 ft., 3 months later, leaving the pipe 12 ft. from the center of the street.

In several instances a cast-iron main laid with cement joints has settled 3 in., or probably more, in loose or filled ground, without developing any leakage. In fact, there is only one case that the writer can recall where the cement joint was not satisfactory. This was in a 6-in. cast-iron main on a dock, about 3 ft. from a railroad track. Many joints in this pipe have developed seepage, and some have small pin leaks, but the leakage is not considered serious enough to warrant closing down the line for re-construction. This failure may be due to faulty construction, as the pipe was laid when the making of cement joints was in its infancy. However, the main is situated so that it cannot be entirely covered.

## Removing Cement-Jointed Pipes.

Fig. 6 shows a completed joint. The cement joint can be taken apart in a very simple and economical way. The pipe is uncovered about one-half, or a little below the center. At the joint where the original bell-hole was dug, the trench is usually made wider on the sides (but not deeper under the pipe), in order to permit the caulker to work at the joint. The upper half of the joint is cleaned out with

a cape-chisel; then, with tripod and blocks, the free end of the pipe is raised until the lower half of the joint breaks free from the bell. The pipe seldom has to be pulled out of the bell, as it nearly always works itself out as the free end is lowered. If portions of the cement stick to the spigot end of the pipe, or fail to be entirely crushed in the bell, it is a very simple matter to clean out the bell with a cape-chisel, or knock the cement from the spigot with a hammer.

On occasions, after a joint has been cemented tight in the line, it is necessary to cut it out entirely (such as for laying a valve on its side; turning a tee or Y in another direction; adjusting a tee to conform to or meet a grade; avoiding a sewer connection or any other unforeseen obstacle). Table 1 has been compiled from records of the actual time spent in doing such work.

TABLE 1.—Time Required for One Man to Dig Out a Complete Cement Joint, Without Removing the Fitting or Gates from the Line.

| Size. | Time. |
|---|---|
| 4 in. | 18 min. |
| 6 " | 22 " |
| 8 " | 26 " |
| 10 " | 30 " |
| 12 " | 38 " |
| 14 " | 48 " |
| 16 " | 60 " |

TABLE 2.—Data Relative to Cement Joints.

| Size of pipe, in inches. | Rings of jute per joint. | Jute per joint, in pounds (approximate). | Number of joints per 94-lb. sack of cement.* | Number of joints per 8-hour day (one caulker). |
|---|---|---|---|---|
| 4 | 2 | 0.14 | 24 | 50 |
| 6 | 2 | 0.19 | 18 | 42 |
| 8 | 2 | 0.24 | 14 | 34 |
| 10 | 3 | 0.43 | 11 | 28 |
| 12 | 3 | 0.51 | 8 | 24 |
| 14 | 3 | 0.58 | 7 | 20 |
| 16 | 3 | 0.66 | 6 | 17 |
| 18 | 3 | 0.73 | 5 | 14 |
| 20 | 3 | 0.80 | 4 | 11 |
| 24 | 3 | 0.95 | 3 | 7 |

* Including the 20% of cement wasted or left over.

The writer has never cut out a joint on a main of greater diameter than 16 in. It is fair to assume that to cut out the upper half of a joint, for the purpose of removing a pipe, would take only one-half the time indicated in Table 1.

At Long Beach unit costs have been kept on all construction, covering nearly the entire 60 miles of cast-iron water mains. Table 2 has been carefully compiled from these unit costs, and presents data concerning cement joints.

The writer has never cut out a joint on a main of greater diameter than 16 in. It is fair to assume that to cut out the upper half of a joint, for the purpose of removing a pipe, would take only one-half the time indicated in Table 1.

[illegible] have been kept [illegible] covering nearly the entire 60 miles of cast-iron water mains. Table 2 has been carefully compiled from these [illegible] and presents data [illegible] joints.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## EARTH PRESSURES: A PRACTICAL COMPARISON OF THEORIES AND EXPERIMENTS

### Discussion.*

By L. D. Cornish, M. Am. Soc. C. E.†

L. D. Cornish,‡ M. Am. Soc. C. E. (by letter).§—Mr. Cain corrects the writer's Sections 15 (Cain) and 23, by stating that, for these cases, where the inner face of the wall lies below his (Cain's) limiting plane, the Rankine method alone applies. Such corrections are justified by his own publications, but the sections as given are correctly computed from the equations derived from the Cain theories, by Professor Ketchum, who, like the writer, probably was unfamiliar with the Cain theory of the "limiting plane". Mr. Cornish.

Mr. Cain discusses briefly the Rankine sections of Figs. 11 and 12, and states that for years he had pointed out the inaccuracy of the Rankine method when applied to walls with vertical backs, the earth being level at the top. He extends this criticism further (E. P. p. 19) by stating his conclusion drawn from experiments "that the direction of the thrust, as given by Rankine, is never experienced in walls at the limit of stability, except where the surface slopes at the angle of repose."

Though the writer's Figs. 11, 12, and 13, Rankine, and Curves 1, 2, 2*a*, and 2*b*, of Fig. 24, do not in any way verify Mr. Cain's conclusions, they certainly indicate the inconsistency and unreliability of the Rankine theory and formulas when applied to walls with vertical backs. For walls with stepped or battered backs, such as Figs. 15

* Discussion of the paper by L. D. Cornish, M. Am. Soc. C. E., continued from November, 1916, *Proceedings*.

† Author's closure.

‡ Cincinnati, Ohio.

§ Received by the Secretary, February 14th, 1917.

Mr. Cornish. and 23, with large angles of surcharge, the Cain analysis, as shown by its author, is inapplicable, and the Rankine method must be used.

Mr. Cain also states that the writer is unwarranted in his assumption that the variation of $p$, in Figs. 2 to 8, offers any criterion for judging of what the variation should be for Figs. 9 to 15, and, to substantiate his statement, offers certain arguments based apparently in part on the assumption that conclusions relative to walls 10 ft. high should not be based on the experimental data of Leygue for retaining boards 8 in. high. If such was his intention, it is due to a misunderstanding of the writer's criticism, as it is plainly evident that the variation in $p$ is entirely independent of the height of the walls. The writer criticised the Cain formulas in only two particulars: one was with reference to Figs. 14 and 15 (Cain), being but slightly larger than Figs. 7 and 8, which is explained by the limiting plane theory, which necessitates the use of the Rankine theory for Fig. 15. The other criticism was that the Cain formula produced a section (Fig. 11, Cain) larger than Fig. 10 (Cain), whereas a smaller section might reasonably be expected because the experimental section, Fig. 4, is smaller than the section shown by Fig. 3. It is not apparent how cohesion can account for such a variation.

The experiments of Dr. Müller-Breslau, cited by Mr. Braune, are extremely interesting, and indicate quite conclusively that the slope of the surface has little or no influence on the slope of the resultant earth pressure. It is to be hoped that Mr. Braune's proposed experiments will develop some valuable data on the vexatious question of the direction of the resultant pressure.

The writer trusts that the paper and discussions have been of some material benefit in emphasizing the necessity for systematic experimentation until sufficient data are obtained to permit the development of working formulas on which all members of the Engineering Profession can agree; and he herewith tenders his thanks to the gentlemen who have so kindly co-operated with him by engaging in the discussion.

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# PAPERS AND DISCUSSIONS



## THE VALUATION OF LAND

### Discussion.*

BY WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.

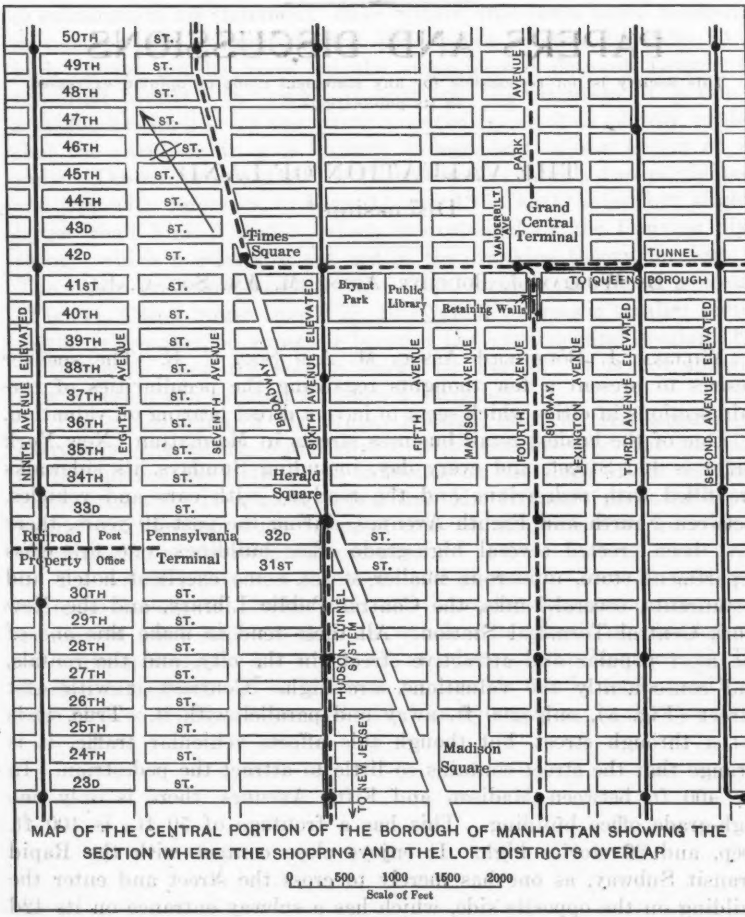
WILLIAM J. BOUCHER,† ASSOC. M. AM. SOC. C. E.—The speaker desires to present a few thoughts regarding the peculiarities of certain sections in cities which seem to have a direct bearing on valuation. Mr. Boucher.

One of the busiest retail business streets in Manhattan (New York City) is 42d Street, and every day, including Sundays, its sidewalks are filled with pedestrians and the roadway with cars and vehicles. Between Fourth and Eighth Avenues, during the past 10 years, there have been erected several high-grade office buildings, one first-class department store, numerous smaller stores, some excellent hotels and restaurants, several banks, the Central Public Library, and the New York Central Terminal Station. All these tend to make this one of the most popular and attractive streets in the city, and the rentals, and consequently the valuations, are high. Contrast it with 41st Street (Fig. 5), only 200 ft. away and parallel with it. True, it is not a through street, but though this affects vehicular traffic, it is strange that the street contains so little to attract the pedestrian. In the 400 ft. between Madison and Fifth Avenues, there is only one high-grade office building. This has a frontage of 50 ft., is 100 ft. deep, and 20 stories high. It enjoys close contact with the Rapid Transit Subway, as one has merely to cross the street and enter the building on the opposite side, which has a subway entrance on its 42d Street front. The remainder of the block is occupied with small shops, restaurants for light, noon-day refreshments and other establishments paying small rents.

* Discussion of the paper by L. P. Jerrard, Jun. Am. Soc. C. E., continued from February, 1917, *Proceedings*.

† Long Island City, N. Y.

Mr. Boucher. Again, consider 43d Street, which is 200 ft. to the north of 42d Street. It is a thoroughfare from the Grand Central Station to the Hudson River, but it has practically no retail, or other business to make large rentals or valuations. It is given over largely to clubs and other buildings where gatherings can be accommodated. One




MAP OF THE CENTRAL PORTION OF THE BOROUGH OF MANHATTAN SHOWING THE SECTION WHERE THE SHOPPING AND THE LOFT DISTRICTS OVERLAP

FIG. 5.

office building fronting on 42d Street, with a subway entrance in front of its door, has an "Annex" building on 43d Street. The buildings are connected by a through corridor giving the "Annex" the full privileges and benefits of the 42d Street entrance, but the rentals are considerably less than those for the 42d Street front. The contrasts

of these two contiguous streets (41st and 43d), so close to 42d Street, form an interesting study. Mr. Boucher.

Reference is made in the paper to the advantages of transportation, and it does seem as though that is the originating feature of value of land, although the speaker makes no claim to being an expert. One location stands out prominently, *viz.*, 149th Street and Third Avenue, and vicinity, in the Borough of the Bronx, which is a goodly distance from "down town", yet, through the influence of good transit facilities (both "Subway" and "Elevated", as well as surface cars), there has grown up in that section in the 12 years since the subway was opened to traffic, a very busy retail business center, where rentals must be quite high. Comparing this location, about 5½ miles from 42d Street, with its good transit facilities, with another location only about 1½ miles from the Grand Central Station, which has until recently had only the poorest, or practically no, transit, and the contrast is remarkable. This other location is in the Borough of Queens, directly across from 42d Street. It is true, it is separated from Manhattan by the East River and, until recently, that stream formed a most effectual barrier to progress. Two ferries, one at 34th and another at 92d Street, gave occasional access across, and brave was the pioneer who had his home or his business there. About 10 years ago, surface car service was established over the Queensboro Bridge from 59th Street, Manhattan, to Queens, but not until the Steinway or Queensboro Subway was opened, in June, 1915, with its free transfer privilege to the main subway line, could it be said that rapid transit existed there.

On the river front opposite Manhattan, from 45th to 59th Streets, there still remain large tracts suitable for wharves and warehouses, and, farther inland, there are other large tracts, with railroad facilities, and now served with rapid transit, simply waiting to be developed with industrial establishments of every kind, with desirable locations for workingmen's homes near-by. These tracts may be had on very favorable terms, and the valuations are low when compared with other localities much farther removed from the business centers of Manhattan.

It is very unfortunate that real estate in the Borough of Manhattan, which, for so many years, has been considered a safe and desirable investment, should have proved so unstable and shifty in the past 10 or 15 years. Since the advent of the modern, steel skeleton building, with its cheap curtain walls and concrete floors, it has offered peculiar inducements for the manufacture of garments of white goods and clothing, in which lines large numbers of men and women are employed. This line of manufacturing, until quite recently, was conducted in the district south of 14th Street, in buildings of the old type so common all through that section. About 10 years

Mr. Boucher. ago, many progressive, but unfortunately, as it proved, misguided owners or speculators seized upon the section north of 14th Street, and, in practically every east and west street, erected many of the so-called "modern loft" buildings of ten and twelve stories, in which these garment trades are housed. The thousands of operatives are mainly recruited from the recently arrived immigrants, and, during the noon hour and at opening and closing hours, the sidewalks and streets literally are covered with the slow-moving, jostling, noisy swarms. At the time this movement of these trades was taking place, an entirely different movement was under way. The department stores which, 15 years ago, were on 14th Street and on Sixth Avenue, north of 14th Street, and on 23d Street, began to move to newer and more commodious quarters in the vicinity of 33d and 34th Streets, Sixth Avenue, and Broadway, and a little later to Fifth Avenue, north from 34th Street. Thus, the new "loft" district and department store district overlapped and conflicted to such an extent that the very existence of the department stores was threatened by the swarms of noon-day loiterers from the "lofts", who slowly wandered up and down and along, completely blocking the passage of would-be patrons of the stores.

The nuisance was so great that the "Save New York" movement has become a reality, and is already bearing fruit. The first result was the legal zoning of the city, during 1916, by which the character of various districts will be restricted, and in the different "zones" certain restrictions will apply, which will prevent the intrusion of businesses or trades into districts which would have their real estate values impaired or ruined by such intrusion. The "Save New York" movement is considerably different from the zoning plan in that it involves a drastic remedy for existing evils in the district bounded by 32d Street (north side) to 59th Street, and from Third to Seventh Avenue. In brief, the scheme of the Committee was: To persuade manufacturers who contemplated moving into this zone to stay out; to induce those manufacturers already in, to move out; and to stop the erection of new manufacturing "lofts" in the zone bounded as described. An agreement was also made by buyers, to give the preference in buying goods to those concerns who do not manufacture in this zone, this measure to take effect on February 1st, 1917. The response to the Committee's appeal was immediate, and up to January 1st, 1917, the following results have been accomplished: First, not a single new lease for manufacturing purposes has been made in the zone; second, not a single new building for manufacturing purposes has been erected in the zone; and third, all but 20 firms, out of a total of 225 now manufacturing in the zone, have agreed to move out at the expiration of their leases, at various times during 1917, 1918 and 1919.

Thus the future of the district is assured by the new "zoning" plan, although tenants of a different class must be found for the buildings already erected which will be vacated. Mr. Boucher.

Referring again to the comment that it is cheap, convenient, and frequent transit, which apparently forms the basis of land values in the first instance, there are many persons who well remember when the districts lying north of 59th Street, Manhattan, were sparsely settled, where rocky hillsides and trees abounded and where goats roamed at will in and around the shanties of the "squatters" who were the first settlers. At that time the means of transit were confined to horse cars and omnibuses, and as the time required to travel from the business sections at or around the City Hall to the neighborhood of 59th Street was fully one hour, it was about all that the patience or the physical health of the passengers could endure, as the cars were not heated and the feet found cold comfort in the straw loosely placed on the floors in winter. The elevated railway system was opened for traffic before 1880, and by 1884 had reached Harlem, and with its advent came the buildings and the people to fill them. A later instance, within the memory of all, is the Washington Heights Section of Manhattan and the sections of the Bronx made readily accessible by the subway opened in 1904. Until that date, only surface cars reached those districts, but during the first 5 years after the trains began running entire neighborhoods were transformed from open vistas and low valuations to closely built blocks of buildings with much enhanced values.

Again, this matter of the influence of transit facilities is very noticeable in Chicago. The four elevated railroad main lines are the Northwestern serving the "North Side" and northwest section, the Oak Park (Lake Street) and Metropolitan serving different sections of the "West Side" and the South Side line serving the southern and southwest sections. Each has a very heavy morning and evening rush-hour traffic. The four lines reach the business section, as shown on the map and there jointly use the "Union Loop" on Wabash Avenue, Lake Street, Fifth Avenue, and Van Buren Street. The "Loop" was opened for traffic in 1897 and, until recently, all trains entering it passed entirely around it and left it at the point of entrance. It was merely a joint terminal, and fares began and terminated there; if a passenger desired to continue his ride, he left one train, went into and through a station, paying another fare and boarded the train of another company also stopping at that station platform. During the past 3 years, however, "through routing" has gone into effect, by which trains are run through from the North to the South Side and *vice versa*, and free transfers are issued from both these lines to the "West Side" lines.

Mr. Boucher.

The "Loop" has been termed both a blessing and a curse. In the first instance it provided a splendid terminal arrangement for all trains, from whatever section, giving preference to none and permitting passengers the maximum of convenience in the matter of reaching the down-town section, but, on the other hand, it has undeniably had the effect of creating the most congested business section in the United States. From Wabash to Fifth Avenues is about 2 000 ft., and between Lake Street and Van Buren Street is approximately 3 100 ft., and within this area of less than $\frac{1}{2}$ sq. mile is concentrated the business center of America's second city. Towering buildings rise on every street, each street is almost equally thronged, although State Street, with its department stores, is noticeably the busiest. Pedestrian, car, and vehicle traffic combine to produce the very limit of impeded traffic. By a reliable count, made by unprejudiced observers in 1910, the number of pedestrians on State Street, passing the corner of Madison Street, was so great that, without doubt, it might be termed the most densely traversed locality in the world. Any fair Saturday afternoon, the 20-ft. sidewalk is thronged with densely-packed, slowly-moving crowds, which fill it from curb to building line, about equally divided between those moving north and those moving south, on both sides of the street.

In this section are the large retail stores, many wholesale stores, some warehouses, the banks, exchanges, hotels, theaters, office buildings, some light manufacturing establishments, in fact, everything that tends to draw people; the result has been that every foot of ground has been utilized by the erection of business buildings, mainly of a very good type, with the corresponding result that valuations and rentals are high.

Step beyond the limits of that "Loop", and on all sides (the East Side excepted) one immediately notices the difference. The buildings are of a different character, and the hurrying crowds are absent. East of the Loop on Wabash Avenue, the city extends one block to Michigan Avenue, overlooking Grant Park and the Lake. Michigan Avenue, like Fifth Avenue, New York City, has been kept free of street cars, its traffic resembles that of the eastern city, except that it has no 'buses, and its development has been very similar. The buildings are of the highest class, consisting of office buildings, first-class jewelry and retail stores, clubs, places of musical entertainment, the Public Library and the Art Institute. Mr. Jerrard's Fig. 1 is a view looking north, as indicated on the map, Fig. 6, showing Grant Park, with Michigan Avenue on the left and Lake Michigan on the right. Marvelous, indeed, have been the changes. The Illinois Central Railroad is shown crossing the pond or arm of the lake on a wooden trestle. All the neighboring section has been filled in, so that the

general level is 14 ft. above the lake, and the park has been enlarged to the extent shown on the map. The Suburban Division of the railroad traversing the park is confined within a depressed way, with Mr. Boucher.

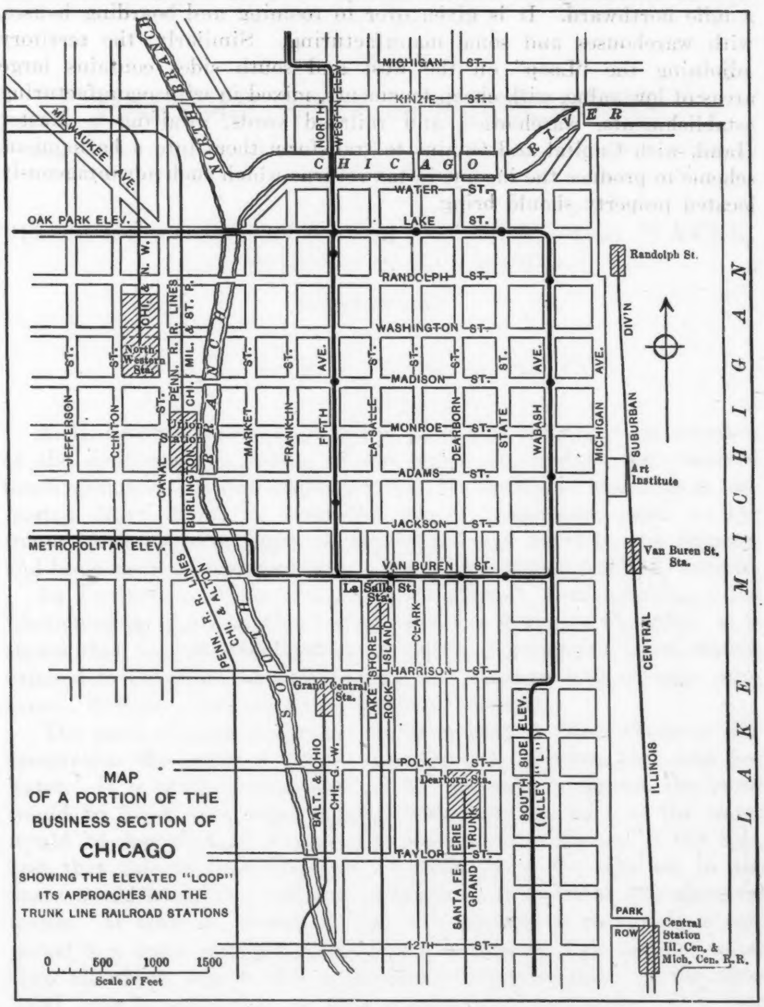



FIG. 6.

retaining walls. On the left, instead of the church spires and comfortable looking dwellings surrounded with lawns, now stretches an unbroken line of high-class buildings giving high valuation to all the section.

Mr. Boucher.

Across the Chicago River (at the top of the map), begins the "North Side". Here, only a step as it were from the busy, high-value center, is a neglected, low-value section extending for approximately a mile northward. It is given over to rooming and boarding houses, with warehouses and some manufacturing. Similarly, the territory adjoining the "Loop" on the west and south sides contains large areas of low value, with cheap tenements, mixed in with manufacturing establishments, warehouses, and railroad yards, awaiting a Master Hand, with Capital and Genius to transform them into a harmonious scheme to produce the income in tax returns which such advantageously located property should bring.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## TESTS OF CONCRETE SPECIMENS IN SEA WATER, AT BOSTON NAVY YARD

### Discussion.*

By A. H. Rhett, Assoc. M. Am. Soc. C. E.

A. H. Rhett,† Assoc. M. Am. Soc. C. E. (by letter).‡—The question of the disintegrating action of sea water on concrete has received much discussion among engineers, and the results of the tests at the Boston Navy Yard, as described by Mr. Bakenhus, seem to the writer to throw some light on the subject and point to one primary and basic cause of disintegration. This is the porosity of the concrete. Mr. Rhett.

In *Technologic Paper No. 12*, of the Bureau of Standards, on the "Action of the Salts in Alkali Water and Sea Water on Cements", it is stated that "a hydraulic cement is readily decomposed if intimately exposed to the chemical action of various sulphate and chloride solutions", if these solutions are of sufficient strength.

The most conspicuous result of these tests is, that whatever disintegration did occur, occurred approximately between high and low water. It is quite obvious that, if the concrete is porous, the pores would be filled with water at high tide, that the salts of the water would be deposited by evaporation during the recession of the tide, and that this process would be repeated until the solution in the pores would become of sufficient strength to give rise to this chemical action. It must be, therefore, that if a concrete is porous, it is subjected to a much stronger chemical action between high and low water than elsewhere, due to this accumulative concentration; on the other hand, if it is non-porous, no such concentration can occur. It is true that evaporation is necessary for the precipitation of the crystals the

* Discussion of the paper by R. E. Bakenhus, M. Am. Soc. C. E., continued from February, 1917, *Proceedings*.

† New York City.

‡ Received by the Secretary, February 27th, 1917.

Mr. Rhett

increase of which in volume causes the actual disruption or disintegration of the concrete, but the unaffected pieces were subjected to the same alternate wetting and drying as the affected ones, and this alone cannot account for the erosion. It seems possible, also, that sea water in its normal condition is not sufficiently strong in sulphates and chlorides to produce chemical action.

The writer has re-stated the conclusions drawn by Mr. Bakenhus on the results of the tests, which conclusions are substantiated by a detailed examination of those tests, and it will be seen that in each case these conclusions can be interpreted as confirmatory of the proposition that the disintegration was more or less proportional to the porosity of the concrete.

"(*a*) That the 1:1:2 mixture is superior to the 1:2½:4½, and that the 1:2½:4½ is in turn superior to the 1:3:6."

It requires no exposition to confirm the statement that the richer the mix the less porous the concrete, with similar conditions of mixing and placing.

"(*b*) That the wet mixtures are superior to the dry."

The tests made by the Bureau of Standards on the hydration of Portland cement, as reported in *Technologic Paper No. 43*, show that there is a latent element which in many cements does not begin to hydrate until 28 days or later, but if then continued in the presence of water, it hydrates with a large increase in volume and with an amorphous structure. It is obvious that this action tends to close the pores of the concrete by filling them with this amorphous material, and that this inert, but most important, action cannot occur except in a wet concrete which is not permitted to dry out too quickly. In the present instance, it was specified that the forms should be "matched" and "as tight as possible" so that the water in the wet mix was retained.

In *Technologic Paper No. 58*, of the Bureau of Standards, on the "Strength and Other Properties of Concrete as Affected by Materials and Methods of Preparation", it is shown from the tests made that, with a given mix, the resulting compression strength of the concrete was about 100% greater with 10.5% of mixing water than with 6.5%; that it was about 25% greater when cured in a moist atmosphere than in the open air; and that the compressive strength is proportional to the density.

"(*c*) That the effects of magnesia and alumina in varying proportions are not very marked, and follow no apparent law, although the two most durable specimens are those lowest in alumina."

This conclusion is confirmed by *Technologic Paper No. 12*, previously referred to, which states that "contrary to the opinion of many, there is no apparent relation between the chemical composition of a cement and the rapidity with which it reacts with sea water

when brought into intimate contact"; and that "the quantity of alumina, iron, or silica present in the cement does not affect its solubility". Mr. Rhett.

"(*d*) That extra care in mixing produced decidedly beneficial results."

The experiments of Michaelis, Cabolet, and others have shown that when cement is continuously mixed with water an increase in volume as much as seven hundred-fold may occur. Such an increase in the volume of the cementitious material would tend to fill the natural voids in the aggregates and make the concrete non-porous.

"(*e*) That hydrated lime was of no benefit, but rather a detriment."

To refer again to *Technologic Paper No. 12,* it is shown that lime is the element of the cement that is attacked by the sulphates and chlorides of the salt water. This action is shown to be even more pronounced with set than with unset cements, the reason for this being that free lime is always liberated in the process of setting. The addition of more lime to the cement of course will increase this action. In this specimen, too, the 10% of lime replaced an equal quantity of cement, and decreased to this degree any possible automatic pore-closing effect from the full hydration of the cement as already described.

"(*f*) That the addition of Sylvester wash was harmful."

The theory of the addition of alum and soap is that they combine and coat the pores with a water-repellant material. It is difficult to see, however, how such an action could occur, and yet the particles of the cement remain uncoated with the same water-repellant material which would prevent their full hydration and the swelling and void-filling action resulting therefrom.

"(*g*) That the addition of clay to the cement had a slightly beneficial result."

It is well known that clay is a colloidal material with respect to water, that is, that it remains suspended in water or holds water in suspension. When mixed with cement, it would seem that this action could not but retard evaporation, and then tend to promote the further hydration of the cement. It is also probable that the clay would have some void-filling effect.

It is affirmed, quite confidently, sometimes, that this disintegration of concrete between high and low water occurs only in northern latitudes, though the writer has never seen any very convincing evidence adduced to substantiate this claim, and that the disruption of concrete is due to the freezing of water in the pores and not to crystallization, neither action can occur, however, without the open pore to pocket the water, and to the open pore must be attributed the primary cause of trouble.

been brought into intimate contact, and that the quantity of aluminate, etc., present in the cement, does not affect its solubility.

"(4) [illegible] of concrete [illegible] beneficial results."

"The [illegible] [illegible] consists of [illegible] of the [illegible] voids in the aggregates and makes the concrete nonporous."

"(c) That [illegible] lime [illegible] but [illegible] a [illegible]."

[illegible] is the absence of the cement that is attacked by the magnesia and chlorides of the sea water. The action is known to be very more pronounced with [illegible] than with [illegible], the reason for this being that free lime is always liberated in the process of setting. The addition of [illegible] lime to the cement will increase this action. In this connection [illegible] the [illegible] of lime [illegible] to equal quantity of cement [illegible] [illegible] already described.

"(c) That the addition of [illegible] with [illegible] beneficial."

The theory of the addition of alum and soap is that they combine and coat the pores with a water-repellent material. It is [illegible], however, that some [illegible] of the cement remain unhydrated [illegible] with the water-repelling material which would prevent their full hydration and the swelling and void-filling action resulting therefrom.

"(d) That the addition of clay to the cement has a slightly beneficial result."

It is well known, that clay is a colloidal material with respect to water, that is, that it remains suspended in water or holds water in suspension. When mixed with cement, it would seem that this action could not but retard [illegible], and also tend to promote the further hydration of the cement. It is also probable that the clay would have some void-filling effect.

It is admitted quite confidently, sometimes, that the disintegration of concrete between high and low water occurs only in northern latitudes, though the writer has never seen any very convincing evidence adduced to substantiate this claim, and that the disruption of concrete is due to the freezing of water in the pores and not to crystallization. Neither action can occur, however, without the opportunity to absorb the water, and so the open pore must be considered the primary cause of trouble.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

### Discussion.*

By Messrs. R. H. Keays, F. Lavis, James F. Sanborn, Lazarus White, T. Kennard Thomson, Francis Lee Stuart, and J. V. Davies.

R. H. Keays,† Assoc. M. Am. Soc. C. E. (by letter).‡—This is a very interesting paper, as it explains new methods for obtaining speed in driving long tunnels in rock. Mr. Keays.

Some years ago it was generally stated that the art of tunnel driving in Europe was much further advanced than in America, as was shown by the great progress in the various tunnels through the Alps as compared with America's much more modest attainments. The proper reply was to argue that the reason American engineers did not specialize in speed was that they had no long tunnels to drive, and when, for financial reasons, they found it necessary to speed up their work they would soon find a way to do so. Certainly, some of their records of late years have been very creditable.

In the writer's opinion, a large part of the credit for the great progress should be given to the improved types of drills developed during the last few years. Where it used to be agreed that the controlling feature of speed was the time taken to drill the heading, it is now considered that tunnel driving is a mucking proposition, and the feature to which most attention should be devoted is the cleaning away of the muck.

In explaining why the pioneer tunnel method was used for the Rogers Pass Tunnel, the author states that the usual "top heading and

* Discussion of the paper by A. C. Dennis, M. Am. Soc. C. E., continued from February, 1917, *Proceedings*.

† Bayonne, N. J.

‡ Received by the Secretary, February 14th, 1917.

Mr. Keays. bench" method was considered too slow and the "bottom heading" method too expensive.

As to the "top heading and bench" method, the writer is of the impression that it has not yet shown its capabilities as to speed, no tunnel driving with modern drills having as yet been attempted where speed was the primary object and where the conditions were so nearly ideal for fast work as in the Rogers Pass Tunnel. The writer has never used the bottom heading method, but has always heard it recommended for its economy. Substantiating this claim, the writer knows of several tunnels where it was used, but they were of such short length that speed was of minor importance.

From a careful reading of the paper, the writer is of the opinion that the author has not demonstrated that the pioneer heading has enough advantages to justify its use as a standard method.

The author gives a list of reasons for using the pioneer heading, showing various advantages which no doubt would expedite work in the main tunnel; but, in the writer's opinion, it would be very little. The paragraphs explaining the location of the portals of the pioneer headings, however, would indicate that, on account of avoiding a lot of soft ground work, the pioneer heading could be driven far in advance of the main heading, but as no heading was driven in the main tunnel, using the pioneer heading to work from, the writer does not understand how greater speed was made in the main tunnel on this account. It merely cheapened the cost of driving the pioneer heading and provided better dumping ground.

The writer assumes that the main heading was being excavated at one face only at any one time, and also that enlargement operations were being carried on at one place only.

Of the reasons given by the author for adopting the pioneer heading, No. 2 is the only one that could tend to expedite work in the main tunnel. Reason No. 1, treating of ventilation, should have no influence on the speed, as the matter of ventilation at the working face can be taken care of very well in the "top heading and bench" method, and ventilation between the heading and portal is not important.

No. 3, of course, would not be an advantage in the "top heading and bench method", for the enlargement operations would be close to the heading.

No. 4 could hardly be called an advantage, for the rate of speed depends on the enlargement operations.

No. 5 is rather unimportant.

No. 6 would also be of no advantage, as the enlargement operations are close to the heading.

Ventilating systems, as usually put in, are disappointing. There is no agreement as to the relative advantages of the pressure and suc-

tion systems, the size of pipe to be used, the pressures, or the quantity of air necessary. Mr. Keays.

It may be said that much larger pipes are required with suction than with pressure systems in order to handle the same quantity of air. The suction system keeps the air of the whole tunnel in relatively good condition, but, at the heading itself, where the men are working, it has the least effect. This trouble is augmented, generally, by the fact that the pipe used is of very light weight and is leaky at the joints, so that there may be very little air taken in at the end.

The pressure system, on the other hand, clears out the heading first, so that the men can go back to work immediately, and ventilation between the working face and the portal is not important.

The pressures generated at the fan are usually so low that a very large pipe is needed in order to get the volume. For economical reasons, a large pipe is always made of light material, which is not satisfactory.

The writer, therefore, on account of the foregoing considerations, believes that a medium pressure system, using a relatively small, screw-joint, standard steel pipe for delivering air to the heading, is the best.

On a recent job he used a Connersville blower, good for a pressure of 10 lb., delivering 750 ft. of air per min. to a heading somewhat more than a mile distant. The pipe was of steel, 6 in. in diameter, and the maximum pressure was about 7 lb. The condition of the air at the heading was always good. The heading and bench were always shot at the same time, every 8 hours, and the blower was usually run for about 2 hours each time.

F. Lavis,* M. Am. Soc. C. E. (by letter).†—This account of the most successful tunneling operation in North America—and this is said with, it is believed, due appreciation of all the work of this kind which has been done in recent years—is a tribute to the advent of the engineer as a director of a type of construction which requires the planning and organizing ability of which usually only the trained engineer is capable. Mr. Lavis.

Confronted with the problems of organization required for the successful economical completion of the long Alpine tunnels, Europeans long ago realized that these were problems for the engineer, and were not of the kind which could be put through solely by "pick, shovel, and pluck," or mere driving power, unaided by a trained mind to plan and execute.

The writer fully recognizes the value of the qualities of the contractor, especially those of the railway contractor, which are very necessary, and which engineers seldom possess. The need of planning,

* New York City.

† Received by the Secretary, February 26th, 1917.

Mr. Lavis. of co-ordinating the work of the organization with the plans, the successful modification of the plans to fit both the conditions and the organization as they develop, are, however, essentially qualifications which the properly trained engineer alone possesses.

Though this is not the first tunnel driven in North America which has demonstrated the ability of the engineer as an organizer along lines which require this necessary co-ordination of planning with execution, it is the latest and most successful, and the writer believes that the realization of this phase, of the conduct of the operations which have resulted so successfully, is of quite as great importance as the recognition of the successful application and improvement in the "pioneer heading" method, important as this latter is as a demonstration of efficiency.

There is little opportunity for comment in regard to the methods used. Figures are not given, though it is hoped they may be supplied later, either by the author or by the officials of the railroad company, showing the value of the time saved and the economy of the extra expense involved in the speed of driving. The writer has no doubt the expense was justified, but the author's comments would be of great value, if he would indicate his views on the economical speed from the standpoint of construction alone without regard to operating losses to the railway, and on the value of the saving to the railroad company.

There is a growing tendency to give, in connection with the description of works of this kind, either exact or approximate costs, and it is rather to be regretted that they have not been stated in this case, as such information has a distinct value, and its suppression is of doubtful utility to the railroad company or the contractors.

The paper is filled with instances which show the careful continuous study of conditions and improvements in details as the work progressed. Many of these latter are not altogether new; some of them have been advocated by the writer and others; but as their general application to tunneling is by no means common, even yet, it seems permissible, even at the risk of repetition, to refer briefly to some of them:

Ventilation is a *sine qua non*.

Bonus system used.

Hammer drills used entirely, columns carefully set with relation to line and grade, holes pointed by clinometer.

Three helpers were used for each two drills.

Fuse was used instead of batteries for blasting.

Water was used to wet down the muck pile and wash the dirt from the sides, roof, and face.

Muckers have brief rest between each carload.

Mucking sheets were used.

There is also one statement full of significance to those who have had much tunnel experience, namely, that "The plant was properly put in and properly looked after, and caused practically no delay". This epitomizes the attitude of those responsible for the conduct of the work, and indicates one of the most important reasons for the success attained. Mr. Lavis.

JAMES F. SANBORN,* ASSOC. M. AM. SOC. C. E.—The speaker has been asked to call attention to a volume, now being prepared regarding the tunnels of the Catskill Aqueduct, recently completed. Dr. Charles P. Berkey, of Columbia University, the Geologist of the Board of Water Supply, is undertaking to collect, as far as possible, information for a comprehensive history of the geological features which had an important bearing on the tunnel design and construction of the several long siphons and other structures of the Catskill Aqueduct. Mr. Sanborn.

It is hoped that this volume will bring out some of the finer points of tunneling. Usually, a tunnel is driven, and one never hears anything more about it, unless, like Mr. Dennis, somebody has the courage to describe it in a paper.

LAZARUS WHITE,* ASSOC. M. AM. SOC. C. E.—The speaker visited Rogers Pass Tunnel in September, 1915, when it was being driven at about its most rapid rate. The profile (Fig. 1), shows that the mountains are only 9 000 ft. high, but, to the speaker, they looked as though they might be 19 000 ft. It was raining at the time; the mountains were covered with snow and ice; and there was snow in the woods; but the tunnel was dry. The work had then been advanced more than 2 miles from the west end and there was just a little trickle of water coming out of the entrance. Mr. White.

The rock was "made to order". It was one solid block, as far as could be seen, practically without faults or the disturbances usually found in rock, and, to the speaker's unpracticed eye, it appeared to be a hard shale or slate. He would hardly classify it as a hard quartzite, as Mr. Dennis does; but he probably knows.

The pioneer heading method appeared to be a very good one for that place, and the whole scheme of the tunnel and its execution a very good piece of work. In the speaker's opinion, it was the first successful combination of the Swiss and the American methods of tunneling. Other attempts have been made to combine the two systems, but they have not been successful, either financially or otherwise. This project, however, was successful in all respects.

The pioneer tunnel was of quite a small bore, and, as Mr. Dennis has stated, it served primarily for ventilation, because through it any quantity of air was sent around the heading, and blew out the smoke. The tunnel was both the driest and the best ventilated of

* New York City.

Mr. White. any the speaker has visited. On account of the extensive blasting that was done, it must be realized that the ventilation had to be well nigh perfect, because from fifteen to twenty rounds were set off in succession in order to break up the rock ahead of the steam shovel.

In any ordinary method of tunnel ventilation, this would have made the work difficult, if not impracticable, but, with this pioneer tunnel, the air was sent around and blown through the main tunnel. There was an immense fan in the pioneer heading through which there was always a strong wind blowing.

The most original part of the pioneer heading has a strong resemblance to the double tunnel of the Simplon. The main heading was more of a center heading than a bottom heading, and, by working within it, the men had plenty of time to do their drilling in the face, which previously had always limited the progress. In this case it did not; the blasting was done as rapidly as the shovel could take the muck. That shovel, of about 40 tons capacity, was worked just as hard as any shovel in the open. It loaded full-sized, 12-yd. cars, and there was continuous and very good service rendered to the shovel, with no interruption.

The speaker believes this progress could not have been made if the rock had not been so exceptionally good, because timbering would have delayed the work, and probably the method would not have been successful if much timbering had been required. Eventually, it will be necessary to concrete more of this tunnel than has yet been done, because the rock may weather.

Another feature, not brought out very strongly in the paper, which contributed very largely to the success of this tunnel, was the drills used. The speaker believes they were little water-fed drills. With these it was possible to maintain the high rate of progress in the heading, and they contributed very largely to the success of the work.

The camp was very well kept, but the speaker was surprised to note that the power plant was of such small capacity. The working organization was very effective, and, without intermission, sustained the undoubted record-breaking progress with which this tunnel must be credited. This organization, the speaker is informed, was brought from the United States, where the men had done some very rapid work on the Gunnison and Laramie-Poudre Tunnels.

The wooden water pipe, used for ventilation, served its purpose well, and was much superior to the ordinary iron pipe. It was well laid, and the joints were so tight that the exhaust could be conducted through it.

Mr. Thomson. T. KENNARD THOMSON,* M. AM. SOC. C. E.—Mr. Dennis deserves the thanks of the Society for his very interesting paper, but it is to be hoped that he will add many details before its final publication.

* New York City.

Mr. Thomson.

The speaker understands that the report of the late Virgil G. Bogue, M. Am. Soc. C. E., is a masterpiece of one of the foremost of old-time railroad engineers, and it would seem that its publication in full, in connection with this paper, would be of great value to other engineers and students. For that reason it is hoped that the author will prevail on the Canadian Pacific Railway Company to allow its publication in full.

Mr. Dennis' paper is of special interest to the speaker because he was engaged on the location and construction of the Canadian Pacific Railway from Medicine Hat to the west slope of the Selkirks in 1883-84-85. Haste, the key-note then, as it seems to be still, resulted once in a record for rapid construction, when 93 miles of railroad from Medicine Hat was built in 23 working days of June, 1883. This work, which included grading, bridges, culverts, and track-laying, but not very much ballasting, really made a marvelous record, considering the fact that all the supplies had to pass through Winnipeg and then be carried some 600 miles over a new, single-track railroad.

In those days no one ever thought that that railroad would ever be a financial success; it was considered merely as a political necessity, as the Dominion Government, at the time of the Confederation in 1867, had assumed the obligation of making physical connections between the separated provinces. Even the late Commodore Vanderbilt had so little confidence in it that he got rid of his holdings.

As a matter of fact, in those days, British Columbia was closer, commercially, to the United States than it was to the rest of the Dominion; and, even in 1885, the speaker found that Ontario or Quebec banknotes were not welcomed in British Columbia and were accepted only at a discount, while United States notes were accepted at par. This was partly due to the fact that in those days the face value of a Canadian banknote was not guaranteed by the Government as it is now. At present, however, the Canadian banking system is superior in many respects to that in the United States where a national bank in a small town has only the credit of its small capital back of it, whereas the smallest branch of some dozen strong Canadian banks has its powerful system behind it. Very few Canadian towns are not amply provided with such branches.

One reason for haste in those days was to get some return on the money as soon as possible, and an even more important reason was the fear that, if the road was not quickly completed, a turn of the political wheel might prevent it from being finished for many years.

In addition to the commercial reasons for haste in constructing the Rogers Pass Tunnel, the fear of snow slides may have been another incentive—the author has stated that the snowfall there is from 30 to 50 ft. a year. The speaker assisted in the design of the

Mr. Thomson. first snowsheds on this road. These were to be constructed in cuts on the hillsides in such a way that the snow slides or avalanches would never strike directly on the roof of the shed, but merely shoot or slide over it. Some of these designs were modified by others, so that the roofs projected up too far, with disastrous results in every case. Trees adjacent to a snow slide would often be cut off as if by a knife about 20 ft. above the ground, or at the surface of the standing snow, by the force of the wind generated by the falling avalanche. Nothing can be built to stop those giant snow slides after they have fallen from 3 000 to 5 000 ft. The snow and ice piled up in winter many feet, and even after the intense heat of August a depth of some 30 ft. or more would be left.

Would it not be appropriate for the author to insert a brief sketch of the life of Major Rogers, one of the most unique characters of a past age of remarkable engineers? He saved the Canadian Pacific Railway from following the Columbia River some 200 miles around—north and then south—by discovering the Rogers Pass which led due west, crossing the Columbia River twice.

The exceedingly favorable geological conditions encountered in this tunnel were not often found in the earlier tunnels in the Selkirks, one of which—a mud tunnel—was nearly completed when it was found that the two headings did not meet by 18 in. This was first blamed on the engineer, but after the lining had been ripped out and replaced, the tunnel at once collapsed. It was re-opened a third time, and again collapsed, after which it was abandoned, and the railway was run around the bluff on a 23° curve. It was operated on this curve for more than 20 years, and then a new tunnel was built much farther from the face of the bluff, where firmer ground was encountered.

In those days some of the grades were as high as 4½%, and negotiating them gave the engines so much trouble that safety switches were generally placed at the foot of the grade, so that if the train ran away it would have to take the switch, which had such a sharp grade uphill that the momentum of the train would rapidly be reduced.

The plan and profile, Fig. 1, shows how the original location wound around the hillsides in order to avoid more excessive grades, and permitting as rapid and cheap construction as possible. The profile also shows why vertical shafts could not be considered, as the hills stand as high as 5 000 ft. above the tunnel.

Mr. Stuart. Francis Lee Stuart,* M. Am. Soc. C. E.—The speaker has read this paper with interest, as the rate of progress was strikingly rapid. As far as the speaker knows, a pioneer tunnel, similar to that used at Rogers Pass, has never been built in the East, although this method is in constant use in coal and iron mines. Usually, the tunnels in

* New York City.

the East have not been so long, have had less cover, and construction progress has been expedited by shafts. The speaker has found it very difficult to compare the progress in tunnels—every one seems to have different conditions which affect the rate of building—in fact, even parts of the same tunnel are not comparable. In groping after a greater speed, a number of new methods have been tried. Mr. Stuart.

In the Stuart Tunnel, one of the most recent built under the speaker's direction, at the Magnolia Cut-Off, on the Baltimore and Ohio Railroad, the tunnel section used was large, being 31 ft. in the clear horizontally, with side-walls plumb to 11 ft. above sub-grade, and a semi-circular arch with 15½ ft. radius, giving a clear height of 22 ft. 11¼ in. at the center of each track.

To expedite matters, the contractor, H. S. Kerbaugh, Incorporated, put in a Marion 28 (⅝-yd.), dipper, air-operated shovel with which he widened the heading, which originally had an area of about 120 sq. ft., and took the bench to an elevation 6 ft. below that of the wall-plate, leaving 11 ft. in bench instead of the usual 17 ft. It effected a very considerable saving in time and expense, as it was possible to widen and place the steel segment at a rate of from 60 to 70 lin. ft. of arch section per week, as compared approximately with 45 ft. by hand, and to enable the remainder of the bench to be drilled without cat-holes. The speaker thinks that matters were about even, as to cost, and, with a little better roof, it should have been cheaper than the old method.

In a further effort to increase the rate of progress in the tunnel, it was decided to use Blaw steel segments to support the roof, in place of the usual timber segments. These consisted of two 7-in., 14¾-lb. channels, riveted together with two ¾-in. rivets, 2½ in. long, with 6¾-in. fillers between, at intervals of about 3½ ft. These segments were divided into three parts; the central portion was 18 ft. 3⅞ in. long, curved to a radius of 17 ft. 6 in.; the end portions were 11 ft. 3⅝ in. long, 5 ft. 0⅛ in. being curved to a radius of 17 ft. 6 in., and the remainder being a tangent to connect with a flanged foot resting on the wall-plate.

The three sections were connected with 9 by ¾-in. plates, 1 ft. 9 in. long, and with ¾-in. fillers. They were used in 2 015 ft. of this tunnel, or for 60% of its length, and, for the most part, were 5 ft. from center to center; otherwise they were 7 ft. 6 in. from center to center, depending on the character of the roof. All were tied together longitudinally with ¾-in. tie-rods 7 ft. from center to center. The lagging was placed directly on these segments, and was fastened through $\frac{9}{16}$-in. holes in the flanges. The sections were assembled in the tunnel and raised to position.

The use of these steel segments enabled the driving to be done more rapidly, as the cross-sections decreased an average of about 2 cu. yd. per lin. ft., and, as the segments came entirely within the section of

Mr. Stuart. the concrete lining, there was an additional saving of practically 2 cu. yd. of concrete per cu. ft.

The speaker's conclusion was that speed was made by the use of these forms—whether or not they were cheaper could not be determined.

Mr. Davies. J. V. DAVIES,* M. AM. SOC. C. E.—The only thing the speaker has against this paper is its great brevity. The author has described a piece of tunnel work which has two peculiar and important features: first, the rapidity of the driving; and, second, the use of a "pioneer" heading.

To have done the amount of driving in the headings which was accomplished on this work for one month, is one thing, but to continue that work at the rates actually obtained, an average speed of 24 ft. a day for 7 days a week in one heading, and for 20 ft. a day for 7 days a week in the other heading, is really a wonderfully fine piece of work.

It indicates very clearly an extremely perfect organization in every department, and the speaker presumes that it is to be understood, from Mr. Lauchli's discussion, that that progress was in part due to the fact that the conditions, as they developed in the driving, and the conduct of the work, were almost perfect. There were no high temperatures; there was no entrance of hot water, as is so common in mountain ranges, and, in fact, very little water at all.

The comparison of that work with the best that has been obtained in the United States and elsewhere is interesting. The only tunnel in America (for which the speaker has figures) which exceeds the Rogers Pass monthly record (and that probably was only for an exceptional month), was the Red Rock Tunnel, built in 1901, in which a speed of 1 061 ft. for the month was attained; but that was in soft rock, where the boring was done with hand-augers.

The other tunnel to which the speaker refers is the Elizabeth Tunnel, on the Los Angeles Aqueduct, where Leyner drills were used in gneiss rock, and there the maximum progress was 641 ft. a month.

In the Roosevelt Tunnel, in Colorado, in granite, the speed was only 435 ft. a month. There, also, Leyner drills were used.

In an extensive undertaking of tunnel construction which the speaker's firm carried out in Mexico, a few years ago, heading progress was obtained, in hard limestone rock, as high as 536 ft. a month, using Leyner drills. The use of these drills has made a very marked increase in the rapidity of tunnel work.

Now, take the foreign tunnels: in the Simplon, in rock of a character probably very similar to this—hard, gneiss rock—a speed of 685 ft. was attained; in the Loetschberg Tunnel a rate was attained, in the north heading, in a hard limestone, of 1 013 ft., whereas in the south tunnel the rate was only 545 ft.; so that this Rogers Pass Tunnel

* New York City.

work stands out pre-eminently as the most rapid that has been accomplished in America up to the present day, and is almost equal to anything that has been done elsewhere. Mr. Davies.

There are various questions which arise from reading this paper, regarding which it would be of considerable value to the membership to have further information, and the speaker hopes that the author may feel willing to go outside of the scope of the paper as presented and give some further information which would add materially to its value.

The particular point of great interest is the value of the "pioneer" tunnel. The author refers to it briefly in the paper as an economical question, and it would be of much value if he could add something in the nature of a balance sheet, having on one side the advantages obtained from the construction of the additional heading, and on the other side the increased cost of driving two heading excavations, as was actually done.

The excuse and reason for a "pioneer" tunnel are unquestionably to increase the speed of construction of the main tunnel, in which there is a large bench to take out, as is involved in the enlargement of such a structure as this double-track railroad tunnel, where the value of the time element, in removing the bench and enlarging to full size, is the main factor involved on the one side of the balance sheet.

There is no doubt that speed can be attained by the driving of a "pioneer" tunnel, and undoubtedly in this case, as the author states, there was warrant for its construction.

Several of those discussing the paper have made remarks as to how the "pioneer" tunnel originated, but it seems to the speaker that any one who has studied the construction of the Simplon Tunnel, as described very fully by Sir Francis Fox,* would be satisfied that the use of the gallery in that work gave the idea from which the "pioneer" tunnel was adopted in this work.

It was used in that work unquestionably for the purpose of ventilation, for transportation and drainage, and to expedite the work.

On the other hand, there was another guiding reason in the Simplon for the adoption of the gallery, as the plans for that tunnel contemplated two single-track railroad tunnels, approximately 55 ft. apart, to be built for east- and west-bound traffic, respectively, and the "pioneer" tunnel was located as the side-wall heading for the second tunnel. It is now being enlarged, or has been in the last few years, to make the second tunnel; but it was located in that position unquestionably for the same purposes as the "pioneer" heading in the Rogers Pass Tunnel was driven, having in mind, undoubtedly, later enlargement for the second tunnel.

* "The Simplon Tunnel", *Min. Proc.*, Inst. C. E., Vol. CLXVIII (1906–07), p. 61.

Mr. Davies. In the Simplon Tunnel the gallery was lined with the permanent side-wall construction at the same time.

In the Rogers Pass Tunnel, as described, there were none of the difficulties which were encountered in the Simplon Tunnel and made the use of a "pioneer" tunnel there so vital to the success of the undertaking.

In the Simplon Tunnel, the "pioneer" tunnel (or gallery, as it was called) was carried right through from end to end, and the gallery was enlarged so as to make a passing siding for trains in the center of the tunnel.

The speaker will be quite interested to obtain from the author at a later date some information as to what pressure he used in the ventilating system, with the Connersville blowers, under the conditions developed in driving this long-distance tunnel. The difference between any short tunnel and one of this length, with no intermediate shafts and no intermediate access, is one of the features of this undertaking which is of very great importance. The ventilation question becomes of the utmost importance, and the pressures at which the ventilating apparatus was operated to produce the forced draft for which the Connersville blowers were used, is of considerable interest.

Another matter on which the speaker thinks some further information would be of considerable benefit to the Society is a statement as to the detailed plan of the bonus system which was adopted for the acceleration of labor, as only a vague reference is made to the subject in the paper. The speaker would also be glad to have a schedule of the rates of wages paid to the men on that work.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## THE WATER SUPPLY OF PARKERSBURG, W. VA.

### Discussion.*

BY MESSRS. JOHN W. HILL, ALEXANDER POTTER, GEORGE W. FULLER, NICHOLAS S. HILL, JR., W. E. SPEAR, T. KENNARD THOMSON, H. F. DUNHAM, AND THEODORE S. JOHNSON.

JOHN W. HILL,† M. AM. SOC. C. E. (by letter).‡—The writer has read with interest this paper on the recent improvement of the Parkersburg Water-Works. Having had the honor of making the original plans for the Parkersburg Water-Works Company in 1881, and later of revising these plans and constructing the original water-works, in 1884, he is naturally more or less familiar with the location and history of this improvement. Mr. J. W. Hill.

The original works, planned by a company of which the late Senator J. N. Camden was a member and the leading spirit in the enterprise, looked almost exclusively to fire protection for the city and the furnishing of water to railroads and manufacturing enterprises; in fact, at that time, 36 years ago, little attention was paid to the hygienic qualities of water supplies in America.

The Ohio River water in its then polluted condition was accepted for drinking purposes by all the cities taking water therefrom. The dangers lurking in sewage were not then well known, and in some respects not suspected. Time, research, and experience have since demonstrated that the Ohio River water in its natural state has not been suitable for drinking purposes for many years, if it ever was.

The Smith system of filtration has been proposed for several cities along the Ohio River and came prominently to the writer's notice while

* This discussion (of the paper by William M. Hall, M. Am. Soc. C. E., published in January, 1917, *Proceedings*, and presented at the meeting of February 21st, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cincinnati, Ohio.

‡ Received by the Secretary, February 9th, 1917.

Mr. J. W. Hill. he was the Engineer Member of the Ohio State Board of Health, when it was proposed as a mode of supply for the City of Portsmouth, a few years ago. At that time, as well as before, the proposition was studied and classed as an artificial sand and gravel filter, and it was thought that such a filter, placed beneath the bed of the river—or rather below low-water stage in the river—beyond reach and examination, during ordinary and high stages, would not satisfy the modern requirements of a filtered water supply.

Plain sand filters, such as are used at Philadelphia, Pittsburgh, Washington, and other places in the United States, and used extensively abroad, are constructed so that they may be under constant observation as to the performance of each particular unit, and when the loss of head has reached a maximum—or other conditions suggest—they can be taken out of service, the sand beds can be scraped and rehabilitated, and the filters restored to their original condition and efficiency. The Smith filters, however, can be examined and repaired only at long intervals, and, if it should occur that the quality of the water obtained from them was unsatisfactory during a period when the stage in the river was considerably higher than the sand bars in which the filters are placed, there would be no relief from the unsatisfactory quality of the water except that which might be obtained from back-flushing the collecting pipes, and that, in the writer's opinion, is always a doubtful method of restoring filters to normal condition.

It was also thought, in connection with the Portsmouth proposition, that, in view of experience with water purification systems somewhat similar to the Smith filter along the Ohio River, there was no assurance, at any time, that the quality of the water would satisfy strict sanitary requirements, and, for this reason, objection was then made by the Ohio State Board of Health to the adoption of this system by Ohio cities.

At several places in the paper Mr. Hall mentions a possible source of water supply to wells from the Ohio River, on the assumption that the water will pass through the silted bed of the river and reach the wells through the sand and gravel in which the latter are driven.

From long experience, with wells along the Ohio River and elsewhere, it is the writer's conviction that wells or galleries sunk below the bed of the adjacent river seldom, if ever, receive water from it. Many tests for hardness, chlorine, and iron, in water from the wells and from the river or stream, have always shown a distinct difference, and, in the writer's experience, this has been sufficient to indicate that the water from the wells is not naturally filtered river water. This experience has been so uniform over a wide range of territory, and throughout a long period of time, that it has led him to conclude that water obtained from wells or infiltration galleries along a stream is

always land-water or ground-water percolating through, or pocketed in, the pervious strata, some of which in due time reaches the stream in the form of springs. In a case where a surface connection is made from the river to the porous material in which wells are driven, the water may percolate through the porous material and reach the wells in this manner, but the writer can recall no considerable success, either from experience in America or abroad, in obtaining large volumes of satisfactory water in this way. Mr. J. W. Hill.

Considering the Parkersburg wells proposed by Messrs. Knowles, Fuller, and Fuertes, the writer would assume that all the supply furnished by these would be obtained from the water moving toward the river from the land side, or pocketed in the porous material along and under the river bed; and though the stages of the river will have an influence on the elevation of this ground-water, it would not imply any flow from the river to the wells.

Several years ago the writer had occasion to test the influence of continuous pumping for 2 months from a gallery or well sunk in a gravel bed along the Iowa River, and, for the purpose of defining the extent of the water field, several observation wells were driven at distances ranging from 100 to 8 325 ft. from the gallery. A level base and bench-marks were established, and the levels of the water in the gallery and in the observation wells before and during pumping were noted from time to time and carefully referenced to the base. The pumping was continuous, day and night, for 62 days, and the elevations of the water in the gallery and in the observation wells, at the beginning, during, and at the end of this period are shown on Fig. 15.

Although the test wells shown on Fig. 15 range from 100 to more than 8 300 ft. in a direct line from the pumping well or gallery, they were distributed east, west, and northwest from it, so as to include the water field which it was thought would be affected by the test. Fig. 15 shows some curious results, with reference especially to observation wells Nos. 10, 11, and 12, all of which were open and penetrated the sand and gravel water-bearing strata, and in which the water levels fell from time to time during the test.

Though the water in No. 12 dropped 1.20 ft. during the 62 days of pumping, at the end of the period the surface of the water therein was considerably above the general elevation of other wells nearest to it in point of distance and in the same general direction from the pump well (northwest).

On the other hand, the water level in Nos. 10 and 11, which were east of the pump well, was at no time as high as in the nearest wells in point of distance, although not lying in the same direction from the pump well. The only explanation for this discrepancy that could be

Mr. J. W. Hill.

given at the time of the test was that the wells were on lower ground and nearer the river, and might have been supplying the river rather than the test pump well. In attempting to draw smooth curves on the diagram for the elevations at the beginning and end of the test, Nos. 10, 11, and 12 were omitted, all the others lying sufficiently near to a smooth curve to indicate the probable water level at the beginning and end of the test.

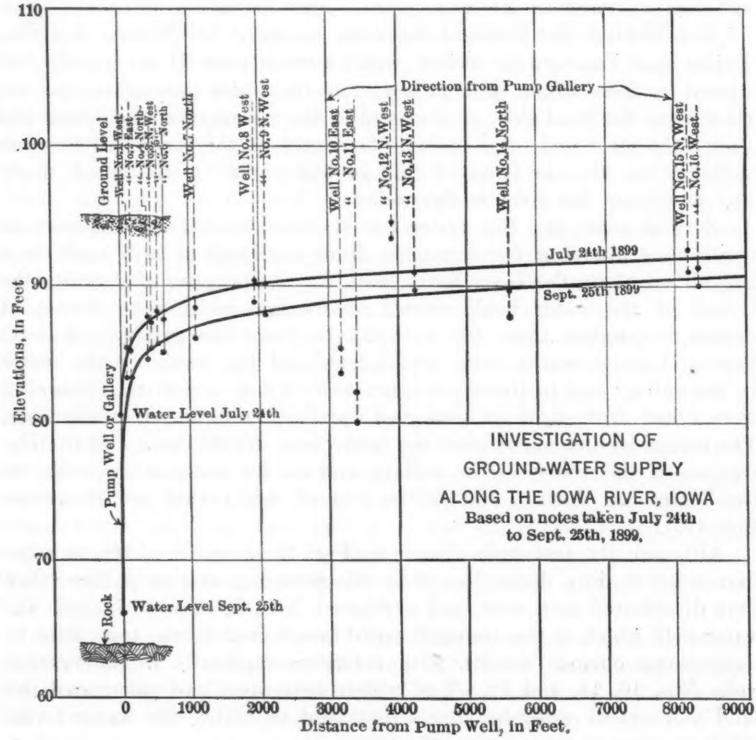


FIG. 15.

"Reference to the diagram, upon which have been plotted the elevations of water in the pump well and observation wells, will show the mean curve of water level which existed in the known area affected by pumping on September 18th. All water in the field above this curve has been pumped away or lost by natural causes, and for the rate of pumpage from the test pump well at that time [693 000 gal. per day] this curve represents the average condition throughout the field.

"The facts obtained from the experimental work [in Iowa] conducted during the summer [1899] are as follows:

"The total amount of water pumped from the test well from July 24th to September 21st, inclusive, was 38 796 662 gal., an average of 646 600 gal. per day. During this interval of time, from the data submitted, the water level in the test pump well was lowered from Elevation 87.78 to Elevation 73.93, or 13.85 ft. Mr. J. W. Hill.

"The total rainfall reported after July 24th aggregates 6.03 in., and, applying this to the area embraced in the investigations, *viz.*, 1 367.4 acres, and allowing an absorption of 40% by the sand and gravel overlying the rock and under the top soil and clay, the absorbed rainfall within the field of observation amounts to 89 555 837 gal., or more than twice the pumpage from the test pump well for the interval of time during which the rainfall was reported.

"The loss of head or lowering of water level from all causes within the field of observation during 57 days of pumping was approximately 2 ft., equivalent to a loss in the field of 235 251 147 gal., to which add the estimated absorbed rainfall, making the total estimated loss from all causes 324 806 984 gal., of which the pumpage from the test well was about 12 per cent."

An interesting part of Mr. Hall's paper relates to the cost of the Smith system of filtration. This is stated as $80 700 for a capacity of 170 000 gal. per hour, or 4 000 000 gal. per 24 hours, or more than $20 000 per million gallons of daily capacity. This is a very high cost for a system of filtration of this kind. Elaborate slow sand filters of the type adopted by Philadelphia, Pittsburgh, Washington, etc., cost no more, and are of such a character that the operation may be observed from day to day, and controlled in capacity and efficiency; and rapid mechanical sand filters would have cost at Parkersburg (in 1911) about one-half as much per million gallons of capacity as the Smith system.

The performance of slow and rapid sand filters can be fully controlled, and maintained at high efficiency, regardless of river stages and other unavoidable conditions which are calculated to operate against sand filters of the Smith type.

The guaranty of 4 000 000 gal. per day for a total area of 1.65 acres is equivalent to 2 426 000 gal. per acre per day, or about the capacity of plain sand filters operating without any preliminary filtration.

The manner of making payments for the Parkersburg filter was unusual. A retention of 55% of the contract price for one year after completing the construction indicates that there must have been a sublime faith, on the part of the contractor, in the final performance of the filter system.

Alexander Potter,* Assoc. M. Am. Soc. C. E.—The City of Parkersburg is to be commended for its courage in putting in operation the so-called natural slow sand filtration method of procuring a Mr. Potter.

* New York City.

Mr. Potter. supply of pure water. This method has been advocated so vigorously by its proponents and has been opposed so bitterly by its antagonists that a plant of this kind actually constructed should prove of a distinct advantage in demonstrating what is of scientific value in the plan and what of a distinctively personal bias on both sides of the much controverted subject.

The speaker has watched with much interest the attempts which have been made from time to time to solve successfully the problem of so-called natural filtration. There can be no doubt that the successful establishment of such a plant will meet with popular favor, and therefore such a project should be given a fair and impartial trial.

The speaker recalls that on an invitation for competitive plans for a new water supply from the City of Evansville, Ind., in 1893, fourteen or fifteen engineers submitted plans. In this competition George S. Davison and W. G. Wilkins, Members, Am. Soc. C. E., were awarded the first prize, the speaker received the second prize, and Arthur S. Tuttle, M. Am. Soc. C. E., the third prize. The plan recommended by the speaker was submerged filter galleries on a sandbar in the bed of the Ohio River, and, as he recalls it, most of the fourteen competitive plans contemplated similar treatment.

Though the speaker's project was not developed by the city, the study made at that time convinced him that, given proper river conditions, infiltration galleries can be designed to give a uniform and pure effluent.

The uniformly good results secured in temperate climates, however, by rapid or slow sand filtration, clearly limit the field of operation of the so-called natural slow sand filtration process, or infiltration galleries, to places where the cost of the latter—the first cost plus operating charges—is less than mechanical or slow sand filtration. In the tropical zone, however, a strong prejudice exists—and rightly so—against the use of stored water, even when filtered, consequently, the development of a supply from infiltration galleries warrants a higher cost than is involved in the construction of mechanical filtration with adequate storage.

Though the analyses of the water included in the paper show a satisfactory effluent, they represent, unfortunately, only a limited period. From the statement made by the author, it appears that radical changes must be introduced in the details of the system in order to prevent a recurrence of the condition found when the beds were examined after having been in service about 3 years. The author states that conical holes were discovered in the sand, having diameters of from 5 to 10 ft. and from 3 to 3½ ft. deep.

It is evident that at all stages of the river, except extreme low water, difficulties will be experienced in maintaining a uniform depth

of sand over the filters, and that, under certain conditions beyond the control of the operator, raw water in large quantities may pass directly into the city supply, thus breaking down the barrier against typhoid fever. If no adequate remedy can be provided, either to prevent blow-outs due to back-flushing, or to restore positively and promptly the sand over the strainers when such blow-outs occur, the system particularly described by the author must be considered a failure, no matter how successful it may be most of the time. Mr. Potter.

With Messrs. Gray, Hall, and others, standing sponsor for the success of the Parkersburg system, it is to be hoped that the best and most intelligent care will be given to its proper maintenance, and that every effort will be made to correct the defects already apparent, which prevent it from producing a uniformly dependable effluent at all times.

A water supply is not built for a few years, or for a decade, but for a lifetime, and because there are those who prefer some other system, no matter what the local conditions may be, engineers should not blind their eyes to the importance of giving a fair trial to this method of water purification.

Passing to the question of infiltration galleries proper, there is no good reason why they cannot be designed to maintain a constant flow and not gradually decrease and ultimately become obsolete, as is generally the case. They should be as dependable as the identical natural process of ground-water seepage into surface streams. An infiltration gallery, to be permanently successful, must never be forced to such an extent as to allow it to be partly emptied. When it is partly empty, high velocities are set up in its vicinity, which, in time, cause the silting up of the filter media immediately surrounding the gallery, which condition slowly but inevitably results in the breaking down of the system.

It is not a difficult matter to regulate the output from a given infiltration gallery so that it cannot exceed a predetermined maximum discharge, and, by so doing, the lowering of the ground-water plane below the top of the gallery can be prevented, which condition will keep entrance velocities within safe limits and thus maintain indefinitely the integrity of the system.

Throughout the country there exist large deposits of sand and gravel which, if intelligently treated, can be counted on to yield large supplies, and there are also sand and gravel deposits in which the depleted water supply can be augmented by the construction of large storage reservoirs over such a system of infiltration galleries. These reservoirs function as feeders to the infiltration galleries constructed within the confines of the subterranean basin.

The maximum rate of downward filtration which can be maintained permanently, is still a mooted question. In the Morris Canal,

Mr. Potter. the leakage, after 100 years of service, averages at the present time 1 250 000 gal. per mile, or from about 200 000 to 250 000 gal. per acre per day. A filter gallery constructed parallel and adjacent to the Morris Canal could be counted on to derive indefinitely 1 250 000 gal. per mile per day from the canal proper. Although feasible to draw off water for short periods at higher rates than this, such an attempt might prove disastrous, as rates of leakage in excess of the natural flow might cause the bottom of the canal to silt up, resulting in a possible break-down in the gallery.

The attention of engineers must be devoted more and more to the determination of safe yields from such underground supplies and galleries, and to the method of controlling the draft therefrom to predetermined and safe quantities.

Mr. Fuller. GEORGE W. FULLER,* M. AM. SOC. C. E.—This is quite an unusual paper. It deals with local politics and personalities, exploits some of the general features of the laws of hydraulics pertaining to underground water supplies, and closes with a description of a filtration project which is unique.

In association with James W. Fuertes, M. Am. Soc. C. E., the speaker reported on the situation, after Morris Knowles, M. Am. Soc. C. E., had been called to Parkersburg to advise as to whether a well-water project was feasible, and preferable to filtration of the polluted and impure water of the Ohio River.

Mr. Knowles' general viewpoint was that a well-water project was a fairly feasible one. That report produced some differences of opinion in the community, with the result that a Board of Engineers was appointed to make further investigations.

The speaker agreed with Mr. Fuertes that a mechanical filtration system was to be preferred to a well-water system. Whatever merit there may be in the Fuertes and Fuller report is not in the detailed results set forth in the paper, but rather in the careful application of the fundamental principles involved.

Effort was made to explain, as engineers, to a community that had its well-defined factions, the underlying principles involved. In outline, the fundamental propositions involved, are first, that, for an adequate underground water supply, it is necessary to determine whether or not there are sufficient porous water-bearing strata; second, where does the water come from, which supplies the strata; third, where, under present conditions, does that water go; and, fourth, what is the best way to collect that water for municipal water supply purposes.

Some rather unusual conditions are found above Parkersburg, in that there is a rocky promontory, a few miles above the city, which

* New York City.

Mr. Fuller.

makes a rock dam across the Ohio River. Consequently, there is no large flow of water in this vicinity beneath the bed of the river. The old river-bed undoubtedly swung to the south, against the West Virginia foot-hills. The surface water from those impervious hills came out above an impervious stratum at the surface of the bottom-lands, and found its way into the river, not through the underground porous strata, but as a surface stream flowing over the impervious clay and silt directly to the river.

Then, it was found necessary to reckon with a partial flow from a very limited drainage area of the river plain, or bottom-lands, plus the underground flow of water to the wells from the adjacent Ohio River. The various hydrographs in the paper show some rather unusual conditions. They show in this river plain a very marked relation between the widely fluctuating water level of the river and the water level in the deep gravel strata into which the wells were driven.

The obvious deduction was that during high river stages there was plenty of water which would flow to wells that might be sunk in the bottom-lands near the river. When the river level was very low, there was not only a gradually receding water-table in the bottom-lands, with the water from the land side making its exit into the river, but there was also the very striking difficulty of having only small volumes of river water flowing back in a reverse direction from the river to the system of wells.

That small increment of naturally filtered river water in the gravel strata was due, of course, to the blanket of silt and clay which formed on the bottom and sides of the Ohio, at times when the flow down the river was insufficient to produce a scouring velocity to push it away.

The ground-water proposition was abandoned for the reason that there was not water in ample quantities coming from the land side, and, as to that coming from the river side, it was very uncertain whether at times of low water the blanket or carpet of clogging materials on the river-bed could be disturbed so as surely to allow water to reach those wells. Mechanical filters were recommended.

S. M. Gray, M. Am. Soc. C. E., was called in to review this proposition, after an injunction suit precluded carrying out the recommendations of the Fuertes and Fuller report. Mr. Gray recommended a project which, although it may have been adopted in certain respects in other communities, was new in its entirety, as far as the speaker knows, in a city of this size. The Smith system appears to be essentially an old-fashioned slow sand filter layout, without any impervious concrete bottom, without any concrete walls, and with reliance placed on two things to maintain the supply during the very varying stages of the Ohio River, ranging from about 5 to 60 ft., from minimum to maximum, in a cycle of a few months.

Mr. Fuller.

During the high stages, the velocity of the flow is so great as to scour away the silt deposits on the surface of the sand bed. To maintain quality and quantity of output at times of low river stages, when there is not a scouring velocity for the removal of surface deposits, Mr. Gray has endorsed, and there has actually been built at Parkersburg, a project with some of the earmarks of a mechanical filter. It has a pipe system scattered over the bottom of 1⅗ acres of sand resting on gravel of one grade, and, at intervals, it is proposed to back-flush one-fifth of that area, or ⅓ acre; and on that ⅓ acre, it is proposed to use about 40 000 gal. during an interval of 15 or 20 min. to remove these clogging materials.

From the standpoint of hydraulics, the speaker is a little disappointed that we have not obtained from Mr. Gray or his assistant, Mr. Leland, some of the details with respect to that proposition. It brings up several queries: In the first place, as regards cleaning mechanical filters, the fundamentals are, in a rough way, that the summation of exit area of the orifices through which filtered water has to pass should be less than the area of the distributing pipe. Otherwise, the proper pressure is not obtained to force water through every orifice throughout that area, so as to lift and wash the sand thoroughly, say, with a vertical rise of wash-water of from 8 to 12 in. per min. At Parkersburg, the sand is not cleaned so uniformly, but the facts as to distribution of wash-water are not made plain.

Of course, it must be recognized that this is not a proposition directly comparable with a mechanical filter, where wash-water is sent at high velocity through a strainer system under every square foot of filter bed in order to give a thorough cleaning. This Smith system seems to be one of those arrangements where back-flushing is used simply to lift the clogging material sufficiently so that whatever flow there may be in the river may be sufficient to move that material beyond this area.

Slow sand filters are back-filled, of course, very slowly, in order to remove air, and Kirkwood's book on filtration states that, more than 50 years ago, such a filter at Tours, France, was cleaned by back-flushing. However, it is stated by Kirkwood* that the head of water was insufficient and the process was not successful.

To build this filtration system of 1⅗ acres at Parkersburg has cost $80 000. That is a somewhat larger sum than was spent at Lawrence, Mass., for building in 1893 an open filtration plant extending over 2½ acres. The speaker does not recall what Mr. Fuertes and he reported with respect to the cost of mechanical filters at Parkersburg, but is inclined to think that it was markedly less than that of the Smith system.

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* Pp. 125–126.

Speaking about costs, mention should be made of one point that is set forth on page 30 of the paper,* where it is stated: Mr. Fuller.

"It is gratifying to note that the new pumping equipment has effected an economy sufficient to pay the capital charges on the bond issue required by the new plant."

As a citizen of Parkersburg and a member of the Water Board, Mr. Hall is entitled to a great deal of commendation if he is able to establish a new and reliable plant without any added cost to the taxpayers; and the low operating expense of the Smith system is certainly one of its strongest points.

This Smith system, which is understood to be patented, has not been exploited at many other places. At Henderson, Ky., it was reported on by Philip Burgess, M. Am. Soc. C. E., and found to have some local merit. George A. Johnson, M. Am. Soc. C. E., and his associates at Wheeling, W. Va., reported on it at great length, with unfavorable conclusions.

It is interesting that, within the past 6 months, Mr. Leland has overhauled the sand and gravel, in connection with back-flushing, rather early in the period of drought in 1916.

With regard to quality, the water furnished by the Smith system is generally of good appearance. In point of hardness, it does not vary markedly from that of the Ohio River. In that respect, it compares with what would be produced by a filter plant of any kind.

With regard to the bacteriological aspect of the water, it is doubtful whether it is of as good a quality as would be afforded by any of the so-called standard modern filter systems or as good a water as the people of Parkersburg ought to obtain. It is rather doubtful whether this water supply is able to comply with the standards of the United States Government, as set forth by the United States Public Health Service, a branch of the Treasury Department, *viz.*, not more than 2 *B. coli* per 100 cu. cm.

That Department has provided certain tests for intestinal bacteria in waters supplied to Interstate carriers. Looking over the analyses on page 65,* it is doubtful whether the City of Parkersburg could sell water to the railroads which pass through it and not get into trouble, if there should be a thorough analytical study of the water.

In making that comment, it is only fair to state that this is a wonderfully transitional age, in point of water quality. Many cities are supplying water of the highest type in point of quality, either by filtration or by the elimination of sources of pollution. By this means the typhoid death rates of American cities have been reduced to a point which seemed almost inconceivable 10 or even 5 years ago. We

* *Proceedings*, Am. Soc. C. E., January, 1917.

Mr. Fuller. have death rates to-day, in cities supplied with filtered surface water, which are as low as those found in European cities, in Holland, and in certain Rhinish districts, where they have pure water from underground sources.

There is, however, a very anomalous situation prevailing in America to-day, in that in some of the surface water supplies there seem to be bacteria of many different kinds, which are classed by laboratory men as members of the *B. coli* group. Some of them are said to be the sour milk bacteria—whatever that may be. Others are said to be bacteria which grow on grain and are found in soils.

In certain districts, where there is a purely rural population, with here and there a scattered village, it is said that there are water supplies which are so polluted, according to laboratory evidence, that they could not reasonably meet the Government standards, even after filtration.

In relation to this matter, it may be stated that there will be published within the next few weeks a new edition of the so-called "Standard Methods of Water Analysis" (American Public Health Association), the purpose of which is to separate some of these forms of bacteria from the so-called *bacillus coli*. Just where it will lead is not known, but the situation to-day, in point of quality of water supply for American cities, is a very anomalous one. It is not readily known whether any given city is living up to the legal requirements of the Federal Government; and, if it is not, it is not known whether the apparent fault is genuinely attributable to sewage pollution, whether it is due to laboratory caprices in reference to the sour milk bacteria, whether it is due to harmless bacteria that grow on grain, or whether it is the fault of the handling of water by filtration, or otherwise.

It is hoped that the next year or so will see a sharp turn in the road, a change from the uncertainty of the past to a definiteness of assignment between cause and effect, so that there surely will continue the improvement in quality which is one of the striking features of the past generation in the point of municipal water supply.

The speaker is glad that the author has presented this paper. It sets forth some simple truths which could be considered to advantage by all who are interested in the subject. He has not been very prolific in setting out the specific details of the Smith system, and it is to be hoped that more data may be obtained later.

Mr. Potter's remarks bring up a question that is very active in the interpretation of the treaty passed in 1912 between the United States and Great Britain, with respect to controlling the pollution of boundary waters. There has been under discussion now for about 3 or 4 years the wording of a procedure to stipulate plainly what shall be

the quality of the boundary waters, so that life or property on one side of the boundary shall not be affected prejudicially by those who discharge sewage into these waters on the other side. After the many efforts made by the Canadian and United States Governments to solve that question, it would be better to wait until we get further evidence from those who are at present studying it. Mr. Fuller.

With regard to Mr. Potter's remarks on the so-called intestinal bacteria, some men want to exploit water analyses very quickly in laboratories, where the results are to be used by the men responsible for the management of a filter plant, or by those who direct the endeavors of a large number of inspectors on a water-shed. Presumptive tests are not well thought of by those who desire to form an opinion on the intrinsic merit of the sanitary quality of water, and especially in the comparison of different waters. For the past 10 or 20 years there have been growing up two factions with respect to applying laboratory methods to the control of water supplies. The men in one set want to get results in 24 hours. They have had their way for the last 5 or 10 years. They have brought into municipal water reports, during the last few years, a set of results which have helped the filter operator, but, to the men who have given years in attempting to control water pollution, these results are open to grave suspicion and are exceedingly difficult, if not impossible, of interpretation.

On considering these figures in type in a municipal report, without any personal knowledge of the environment to which these data apply, trouble ensues. Some of these bacteria, classed as *B. coli,* innocent as they may be, may multiply and breed in water under certain temperatures. This certainly adds to the complications. There ought to be a marked change with respect to laboratory data for the sanitary control of the quality of water supplies in America. Engineers must recognize two distinct phases, one, the quick result to help the filter operators and the men who direct inspectors. When it is desired to get a measure of sanitary quality, however, it is necessary to go deeper and further, and to take 8 or 9 days for analysis, something which the filter operator will not tolerate. On the proposition of telling whether or not a water will measure up to prescribed sanitary requirements, it seems highly important to make every effort to separate fecal from non-fecal bacteria in the laboratory.

Nicholas S. Hill, Jr.,* M. Am. Soc. C. E.—Mr. John W. Hill and Mr. Fuller have covered this paper so thoroughly that it leaves very little to be said in the way of discussion. The speaker certainly agrees with both of them in their criticism of the Parkersburg system. Mr. N. S. Hill, Jr.

* New York City.

Mr. N. S. Hill, Jr.

The situation in Parkersburg recalls a condition that so often exists, and the futility of expert reports under certain conditions, that is, the natural tendency of people to adopt a so-called pure underground water supply, with all its uncertainties as to quantity, rather than the acceptance of a surface water supply of known abundance, treated with modern processes which have a known efficiency.

The use of underground waters is not without dangers; and, without discussing this paper in detail, it might be well to call attention to a situation which sometimes arises where underground water supplies are developed. The speaker alludes more particularly to the East Orange water supply in New Jersey, and will refer to the case of Harper *versus* the Mountain Water Company. This case was tried at Somerville, N. J., before the Hon. William J. Magie, then Chief Justice.* The following is a passage from the Judge's charge:

"I think I should first give you what the law defines as the rights of parties respecting water running in streams. Any one owning on the banks of a stream of running water is entitled by law to have the waters of the stream come down to him in their natural flow, uncontaminated, unreduced, and unobstructed, except so far as Nature affects them. Persons owning above the line of the person who has these rights have also rights in the water passing them, and their right is to have the water pass them and go down to those below unaffected by obstruction from below or unaffected by obstruction or abstraction from above; each of these parties has a right to the use of the water, which is a reasonable use of the water as it passes. Water may be thus used for irrigation, for the watering of cattle, for the use of water around buildings and houses, for the use of water to produce power and thereby run mills or whatever else power may be applied to. I say the riparian owner, that is, the owner on the bank of the stream, has the right, the right to water coming down from above unobstructed, unpolluted, and in its natural flow."

The same rule has been applied to a clear case of a subterranean stream, and apparently no distinction is drawn, whether the underground water is regarded as percolating water or water of a well-defined subterranean stream.

Vice-Chancellor Magie proceeded to say that though a person or company might strike underground streams and abstract underground water which "might" come to the surface, and might carry this percolating water off and sell it, yet, if by their works they abstracted the water or reduced the level of springs or streams which "had" come to the surface, it was an actionable wrong. The words "might" and "had" in this case are emphatic. The jury was directed that, if it was shown by a preponderance of evidence that the defendant had abstracted an appreciable quantity of water which "had" come to the surface in a pond, spring, or stream, so as to diminish the flow, and

* A convenient summary of this case, by Vice-Chancellor Emery, may be found in 20 Dickinson, p. 479.

had done this either by pumping from the stream or from a well, or by both, then the plaintiffs were entitled to damages. Mr. N. S. Hill, Jr.

Later, there has been the decision in the case of Frank Meeker *versus* the Mayor and City Council of East Orange. The City of East Orange recently established an underground water supply, deriving its water through wells about 150 ft. deep. The plaintiff stated that, before and at the time of the establishment of wells, he owned certain property and lands in Millburn and Livingston, having an area of about 100 acres, that by reason thereof he was entitled to the benefit and advantage of a certain spring and stream running and flowing into and through those lands, and that the defendants, by artesian wells, pumps, and reservoirs, did, between January 19th, 1905, and September 25th, 1906, divert large quantities of water from said springs and streams, to the prejudice of the plaintiff.

Judge Adams, before whom the case was tried, charged the jury as follows:

"If you are satisfied that if, by the operation of the defendants' works in intercepting underground waters, any existing natural surface stream was diminished in its flow on the land of Mr. Meeker to an appreciable extent, he has suffered an actionable wrong, and is entitled to compensation."

Furthermore, he stated that the alleged diminution of the surface streams and the alleged destruction of the run-off from the springs are claims independent of one another, but covered by the same legal principle. He referred to the Harper decision, and stated:

"It is not easy to determine just what inferences are legitimate from the language used by His Honor, Chief Justice Magie, in dealing with a case similar to this and yet not altogether analogous. On the whole, I conclude that the Harper case warrants this Court in saying that when an existing spring, actual, not merely potential; visible, not occult; a spring, not a well; brought to the surface by natural force, not conducted there by the labor of man—when such a spring is depleted by subterranean suction occasioned by the operation of a pumping plant like that at East Orange, an actionable injury is inflicted upon the owner of the land on which the spring is situated, for which the law will award sufficient compensation."

In another case, Smith *versus* the City of Brooklyn (160 New York, page 357), the following principle was laid down. If a city, by the operation of a water system, consisting of wells and pumps on its own land, drains the contiguous territory, and thus diverts and diminishes the flow of water in a natural surface stream on the land of another, it is answerable in damages, under the law that no one may divert or obstruct the natural flow of a stream for his own benefit to the injury of another.

Under these decisions, no provision whatever has been made for the assessment of permanent damages. A man brings an action for

Mr. N. S. Hill, Jr. the damages resulting from the diversion of underground waters for a definite period, and asks to be reimbursed for the damage which he has suffered during that period. With surface streams the damages awarded are permanent, and the Courts have defined their measure as the difference in the value of the property before and after diversion. The result of these decisions has been that East Orange has been in constant litigation since 1905, when its underground system of water supply was started, and the City has been compelled to acquire by condemnation more than 1 750 acres of land for which it has no specific use, in order to prevent continuing damages from accruing from year to year. This property has cost the city $400 000 more than the original estimate for the water supply system, or approximately 25% of its value.

There are many situations in which such a condition may develop, and every engineer should use great caution in advising a municipality to embark in an underground water supply before satisfying himself that such a condition is not likely to arise.

The speaker simply calls attention to this added element of uncertainty, in conjunction with consideration of this paper. It is not always possible to develop underground water supplies and be entirely free from claims for damages to surrounding property, as many suppose.

Mr. Spear. WALTER E. SPEAR,* M. AM. SOC. C. E.—It would be pertinent to the discussion of this paper to speak of two of the larger German ground-water works which derive at times a large portion of their supply from adjacent streams in somewhat the same manner as the Parkersburg works. The collecting works of the first of the two large ground-water plants supplying Dresden—the Saloppe Works—with which many members of the Society are doubtless familiar, consist of a slotted cast-iron pipe, about 4 700 ft. long, and from 18 to 26 in. in diameter, which is laid along a low bank of the river about 50 ft. from the edge of the water at ordinary low stages in the Elbe, and about 8 ft. below the river surface. The cover of sand and gravel over the gallery is about 15 ft. The maximum yield of this water in 1904 was given as 12 000 000 gal. per day. Originally, it was believed that all the water obtained at this plant represented the ground-water flow from the upland water-shed that came slowly through the gravel strata in the broad river valley. This idea, however, was soon abandoned. At high river stages, when the ground over the gallery was flooded, it was found that very considerable quantities of water came directly from the river through the 15 ft. of cover. Ordinarily, the water from this gallery was of very good quality, but, when the river was in flood, and a large proportion of the supply was filtered river water, it showed some deterioration. During a

* New York City.

flood in 1896, when the river was 10 ft. above the surface of the ground over the gallery, there were several thousand bacteria per cubic centimeter in the water delivered at the station. There was no typhoid fever in Dresden as a result of the flood, but many cases of intestinal disorders were reported in the city. Mr. Spear.

The Schierstein Works, supplying Wiesbaden, at the time of the speaker's visit in 1904, drew a supply of water from 32 wells driven along the Rhine at a distance of from 200 to 400 ft. from the water at low river stages. These wells were about 35 ft. deep, and penetrated a thick stratum of sand and gravel, which was covered for the most part with a layer of sandy clay 10 ft. or more in thickness. Ordinarily, the supply from these wells was of good quality. It was practically all ground-water, but the cover of sandy clay was known to be absent in the river channels, and the director of the works, Mr. Halbertsma, considered it wise to put in an ozone plant at the works to sterilize the water when the Rhine was in flood.

In looking over the description of the Parkersburg collecting works and noting the meager thickness of cover over the screen sections, it would seem to be the part of prudence for those responsible for their operation to put in a sterilization plant. Although such a plant might well be operated all the time, it should certainly be used when the river is in flood, when the surface of the filter, therefore, is inaccessible, and when the analysis of the water shows signs of pollution.

The Parkersburg plant has some slight resemblance to artificial ground-water works constructed extensively in Sweden, where there are natural beds of sand and gravel, as at Parkersburg, without any adequate supply of ground-water. At these works in Sweden surface water is applied through open channels to the surface of these beds of pervious material, and the water filtering through is abstracted below, generally from wells driven at a safe distance from the open channels. The surfaces of these beds may be cleaned like a filter, by cutting off the supply and allowing the water to drain away. In respect to the depth of material through which the water flows to the wells, and the readiness with which the filter surface may be cleaned, these Swedish works differ from the Parkersburg plant, and these points of difference seem to constitute the chief elements of weakness in that plant.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—This paper is of special interest to the speaker because he has long had a high opinion of the author, a graduate of West Point, and has himself had considerable experience in, on, and with the Ohio River, first at Kenova, about 125 miles south of Parkersburg, where the depth was 6 ft. at Mr. Thomson.

* New York City.

Mr. Thomson. low water and 66 ft. at high water, with very sudden fluctuations or floods; then at Mingo Junction, about 75 miles above Parkersburg.

The speaker also designed caissons for the Wabash Bridge over the Monongahela River at Pittsburgh, just above the junction of the Monongahela and Alleghany, about 200 miles above Parkersburg.

Mr. Fuller has mentioned the impurities of the Ohio River water, and his reference can readily be understood by any one who knows the river. In the first place, to give some faint idea of the sediment carried by this river, it might be stated that, during the winter of 1890-91, one of the coffer-dams for the Kenova Bridge, which was approximately 30 by 100 ft. and 25 ft. in depth, was left in the river. When the waters subsided in the spring this coffer-dam was found to be filled to the brim with sediment. The speaker has no means of knowing whether or not the river could have filled it more than once during the winter.

A river which carries such an immense quantity of sediment and is subject to such severe floods and dry spells is not very dependable, to say the least. To make matters worse, the river is not only polluted by the discharge from many sewers, but, also, during these periodical floods, by the water washing and receding from the streets, cellars, houses, stables, barnyards, chicken coops, etc., of many towns and cities, Pittsburgh included. In fact, one would imagine that the water would be much better for fertilizing crops than as a domestic supply.

The speaker thinks the Parkersburg authorities were wise not to try to pump 3 000 000 or 4 000 000 gal. of water a day from fourteen wells placed some distance from the shore; for he does not think it possible to pump such a volume of water continuously without removing more or less sand, perhaps in almost imperceptible quantities, but still sufficient to form cavities.

These cavities would probably cause no damage in the open fields, but, sooner or later, these fields will be built on by the city; and though the subterranean cavities might even then give no trouble for many years, they might sometime, suddenly, cause very serious collapses, similar to those which have happened to cities built over coal mines. The experience gained by Parkersburg through its adopted plan would seem to prove the correctness of this assertion.

Some day all the towns along the Ohio (and this applies also to other rivers) will combine to dam the water up in the mountains, where it is comparatively pure and free from sediment, and pipe it to the different towns as needed. This will not only give an adequate and dependable supply of good water, but will also regulate the flow in the rivers, doing away with the disastrous floods of periodical occurrence. As an example of many towns along the Ohio: when the speaker first

saw Catletsburg, Ky., the town stood on a bluff some 20 ft. above the river. The next spring he saw rowboats landing at the second-story windows of the houses. These floods occur so quickly that, even with the excellent modern Weather Bureau service, the inhabitants do not always have time to protect their property; yet they come back, clean their houses, make the best of their losses, and start over again; they very seldom move away or build on high ground. Mr. Thomson.

The Court decisions quoted by Mr. Hill would seem to indicate that the small towns may some day be able to force the towns above them to provide pure water or to stop polluting the rivers.

H. F. DUNHAM,* M. AM. SOC. C. E. (by letter).†—A paragraph on page 58‡ giving the dimensions of the intake pipe could be illuminated by mention of the connections to, and the character of, the pumping machinery. It would appear from the text that the large diameters of the intake or "suction pipe" were expected to lessen the otherwise "undesirably rapid rate" of flow through the filtering material. This implies the use of intake pipes as reservoirs from which a certain quantity of water could be taken without marked or legitimate effect at the strainers. Is this view correct? Ordinary experience would tend to show quite the reverse; that the friction losses would be less and the draft at the wells greater with large pipes under a vacuum produced by pumps of any description. Mr. Dunham.

Strainers with V-shaped openings have been used by the writer, not always in vertical positions, but always at some angle and with connections that would allow of access to each well or to each one in a small group when shut off from the others in a system. Any other method seems to be open to distrust, although it is true that conditions vary greatly, and each case is a problem by itself.

Many years ago, the writer examined sand bars in the Ohio River in the vicinity of Wheeling, W. Va., for the Wheeling Water Board, but found little encouragement in quality or volume, although the beds were continuous to a depth of 40 ft. below low-water levels. Besides hardness, the figures for iron were too high, and filters were recommended.

THEODORE S. JOHNSON,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—In reviewing the problem of water supply, the same questions that arise at the beginning of the study are still to be considered after some one solution is adopted; namely, is the water obtained of satisfactory quality from chemical, physical, and bacteriological standpoints, and, is the requisite quality obtainable at a fair cost of production? Mr. Johnson.

* New York City.

† Received by the Secretary, March 6th, 1917.

‡ *Proceedings*, Am. Soc. C. E., January, 1917.

§ Granville, Ohio.

‖ Received by the Secretary, March 8th, 1917.

Mr. Johnson.

Usually, these questions can be answered after some investigation. It is to be regretted that rarely are two solutions of a water supply problem provided so that they may be compared in actual performance under the same conditions of demand and production and at the same time. In this discussion the operation of one type of supply is presented, and an attempt is made to compare it in results with one not in operation. This is a handicap in solving the problem as to the scheme which has not been adopted.

This paper recites the preliminary investigation for a well supply for Parkersburg, and then describes the construction and operation of a system not capable of such preliminary investigation, concerning which little or no previous research or study had been made. It would have been desirable to have answered in the paper the following questions:

1.—How does the present supply compare, in turbidity, total hardness, iron and bacterial content, in color, and in other factors affecting the desirability of a water supply, with the same factors and qualities of the water taken from the well supply as investigated by Mr. Knowles and by Messrs. Fuertes and Fuller, which supply was rejected?

2.—How does the cost of pumping from the present beds compare with the estimated cost of pumping from the line of wells? Speaking more generally, how does the cost of operation, including capital charges, compare with the same cost as estimated for the rejected supply?

3.—What may be the probable life of the present plant, as compared with that of the plant for well pumping? The object of this question would be to show the effect of depreciation and maintenance charges on the cost of pumping.

The data in the paper are not sufficient to enable the reader to determine the answer to the first of these questions. The reports published in the paper, showing turbidity, alkalinity, and total hardness, are given only for one month, and therefore cannot be regarded as showing very fully the characteristics of the new supply. The figures in the table on page 63* are interesting. For example, the iron content of the raw river water varies from 1.06 parts per million, on January 20th, to 7.04 parts per million on January 24th, 1914. This is a very wide range for iron content in the raw river water in such a short period. Again, the average iron content for the month is 3.33 parts per million. Having been familiar with the untreated Ohio River water from his earliest remembrance and being at present a user of water in which the iron content is slightly more than 1 part per million, the writer is surprised to find the Ohio

* *Proceedings*, Am. Soc. C. E., January, 1917.

River water so contaminated with iron. He is still more surprised to find, on page 61, a report of an investigation of the supply by Mr. I. L. Birner, which states: "The total iron in solution, of the filtered water, was 0.19 of one part per million, against an average of 5 to 16 parts in the river." From the wording of Mr. Birner's report, one is led to believe that he refers to parts per million, and this would seem to indicate a river water extremely polluted with iron. The only data which the writer has immediately at hand are those given in the Report of the Ohio State Board of Health for 1908, on page 206, where results are given for tests made at Marietta, Ohio, a few miles up stream from the site of the present filtration works. These tests show an average iron content for 1906 of 0.62 part per million, and the maxima for the year are found to be 1.00 and 1.23 parts per million for August and November, respectively. It is difficult to reconcile these two statements. Mr. Johnson.

In the same report the bacterial results show the untreated water at Parkersburg to have from 100 to 3 100 bacteria per cu. cm., and the filtered water from 24 to 1, with averages for the month of 900 and 9.3 for the untreated and filtered waters, respectively. Tests by the State Board of Health at Marietta show that the average of a large number of determinations indicates a bacterial count of 8 200 per cu. cm. in the raw water during 1906, and 7 350 during 1907. The same investigators, in tests made on July 24th, 1906, at Pomeroy, Ohio, show 19 100 bacteria per cu. cm. in the raw river water and 8 200 on November 19th, 1906. Although these are only scattered results of tests, they serve to cast some comparative light on the published figures. That the safe condition of the water, as indicated by the tests of Dr. Rose and Mr. Birner, is not generally recognized, is shown by the suggestion that a report prepared by the City of Wheeling and tests by the State Bacteriologist of West Virginia both agree that at times the water is quite unsafe.

Engineers may then reasonably inquire whether experience with other well supplies, or tests on the wells as driven at Parkersburg, would not indicate that the quality of the water from the well supply is better than that furnished by the present infiltration system. If there are published or available reports on the well supply, or additional tests on the present supply, which will throw more light on this question, it would seem highly desirable to have them published.

To answer the second question—how does the cost of pumping from the present beds compare with the estimated cost of pumping from the line of wells?—the necessary information is not available in the paper and, by all means, should be presented by the author or by the engineers concerned. The figures on page 59, showing the cost to be $11.03 per million gallons, are very evidently only for the

Mr. Johnson. cost of operation, and include no capital or depreciation charges. The data on pages 58 and 59 do not separate items in construction cost in such a way that direct comparisons can be made.

The service life of the present plant is necessarily one of doubt. It is not the time in which the present construction will furnish the proper capacity, with which we are concerned, but rather the life of any one filter unit. Relative to this it is noted that an examination of the beds in 1916 showed that, at several places, there were holes in the sand and gravel covering. These holes were from 5 to 10 ft. in diameter, and were formed like craters, extending to within 1 or 2 ft. of the brass strainer pipes. The writer has been long familiar with the locality in which the filter beds have been placed, and was engaged on some surveying work, in connection with the quantities of excavation under the contract, at a time very near that at which the photograph shown in Fig. 13 was taken. At that time, he was able to walk to the coffer across the slightly raised level of the bar, and, on every side, were to be seen similar crater-like holes in the natural surface of the sand-bar. In some of these there were pools of water in which children were bathing. It has been the writer's opinion that if these craters would form in the natural bed, they would be as likely to occur over the filter bed, and, if so, would of course greatly impair its efficiency. Though these holes may be refilled, it would seem hardly possible to conduct the refilling so as to secure uniform stratification in the bed, and the reasonable life of the infiltration plant would be considerably shortened by this cause alone. The efficient life of a rapid sand filter or a slow sand filter is a question which can be answered with fair accuracy. It would not seem that the conditions providing stability in the plant were to be found in the shifting variable sand bottom of the Ohio River.

In a general way, the comparison of this plant with either a filtered water supply from the Ohio River or a supply drawn from wells in the gravel beds of the adjacent bottom-lands can be put on a satisfactory basis only when the Smith infiltration system is regarded as being of the caisson type or as a type of sand filter. If of the latter type, it would seem that the same conditions which have been regarded as necessary to secure a satisfactory supply from a rapid sand or slow sand filter would have to be applied to the Smith system. Among the first requisites for either of the usual filter types is a careful control of the operating head, as affecting the rate of filtration, and both types are considered as sensitive to small changes in the effective heads. With a range of 53.1 ft. in the stage of the river, it would hardly seem probable that the filtering conditions remain the same.

Mr. Johnson.

Another requisite of proper sand filtration is a careful control of the cleaning processes. The filters are constantly inspected and watched, and the conditions carefully maintained for securing equal fitnesses and thorough washing of all their parts. If the beds of the Smith system are pitted with crater-like holes extending to within a short distance of the strainer pipes, the efficiency of the back-flushing would certainly be reduced. If, as a matter of experience, such back-flushing does cleanse all parts of the filter bed very thoroughly and uniformly, the data which prove this fact would be interesting.

With a supply of extremely varying nature, such as that found in the Ohio River, sand filtration plants are accustomed to have constant and rigorous tests at least daily, and often more frequently, on the basis of which the rate and character of their operation is altered. Although the author recommends systematic tests, these tests will be lessened in value by the fact that little or no change could be made in the operating conditions of the filter system. It may then be concluded, in the writer's opinion, that the Smith infiltration system may not be regarded as a purification plant, capable of providing at all times a safe and wholesome supply of water such as is obtainable from rapid sand filtration plants, and, therefore, must be considered as an underground caisson or ground-water collection system, of rather high construction cost, drawing water either from the river above or the ground-water below, and confining its purification processes to mere straining.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## DISCUSSION ON PROGRESS REPORT OF SPECIAL COMMITTEE ON FLOODS AND FLOOD PREVENTION*

By Morris Knowles, M. Am. Soc. C. E.†

Morris Knowles,‡ M. Am. Soc. C. E. (by letter).§—In contributing to the closure for the Minority, the writer expresses the conviction that the character of the discussion presented demonstrates conclusively that the attempted premature discharge of the Committee, on two occasions, and before a final report was submitted, was an unfortunate mistake. It is evident, also, that the discussion has fully justified the contention of the Minority that assertive statements, with a limited point of view only, and in the face of present divided opinion on the subject, will not promote progress or the development of a much needed comprehensive programme for stream control, but will rather intensify discord where it is not necessary. The object of the Committee, in so far as it could not agree on a final pronouncement, ought to be to promote further investigation and discussion, and not to attempt to close the door by dogmatic expression of a single point of view. Mr. Knowles.

It is apparent, at a time when the importance of regulation of stream flow in its largest sense, which includes flood control as a part, is engaging the attention of the country and impressing itself on laymen, that the Society has lost a wonderful opportunity to promote sound thinking and the development of a programme looking toward wise legislation. It is plain from the public interest, general

* Continued from January, 1917, *Proceedings*.

† Minority closing discussion.

‡ Pittsburgh, Pa.

§ Received by the Secretary, February 15th, 1917.

Mr. Knowles. discussion, and proposed laws, that no one type of control is a universal panacea, or will do for all classes of streams; also, that the opinion of a single branch of scientific men, not in sympathy with all phases of stream regulation, will long satisfy the country, for such opinion will stifle progress and unduly lengthen the period required for reaching the desired results.

The writer expresses the hope, therefore, that, at some later date, the Society may take up this important subject again and continue its consideration until it becomes possible to prepare a report which will merit the consideration justified by the high standing of the Society.

**AMERICAN SOCIETY OF CIVIL ENGINEERS**

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# PAPERS AND DISCUSSIONS



## DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By William Fry Scott, Assoc. M. Am. Soc. C. E.

William Fry Scott,† Assoc. M. Am. Soc. C. E. (by letter).‡—There are many reasons why this Society, as a body, should not approve the Final Report of this Committee as representing an expression of "the present state of the art." One reason will be sufficient to prove this statement, and may be understood by a study of the diagram, Fig. 6. Mr. Scott.

This diagram gives the variations in the modulus of elasticity (so-called for want of a better term) of embedded steel, in two of the several groups of reinforced concrete beams made and tested under the direction of Mr. Humphrey, the Secretary of the Committee, and, as computed by the writer on the basis of Assumption *c* 4, page 1681,§ of the report. On the other hand, the modulus of elasticity of naked steel is assumed, by the Committee, to be identical with that for embedded steel; as is also the case, by the writers of the paper‖ reporting the beam tests referred to.

This paper has a dual character: On the one hand, it contains the most valuable report of, probably, the most comprehensive series of tests of concrete and reinforced concrete to date. The data are presented in such a thorough manner that they will serve for analysis by a

* Continued from February, 1917, *Proceedings*.

† Dunnville, Ont., Canada.

‡ Received by the Secretary, February 9th, 1917.

§ *Proceedings*, Am. Soc. C. E., December, 1916.

‖ Published by the Bureau of Standards, *Technologic Paper No. 2*, "The Strength of Reinforced Concrete Beams; Results of Tests of 333 Beams," by Richard L. Humphrey and Louis H. Losse (June 27th, 1911).

Mr. Scott. stranger to the experiments just as well as they did for those who planned them. On the other hand, it contains an elaborate thesis that is in no way supported by the data in Tables 3, 33, 34, 35, and 36, and Appendices I and II, of the paper. This thesis follows more or less closely the "design" assumptions on page 1681 of the report of the Committee, and the authors seem to have lost sight of the fact that their real object was to investigate the correctness or falsity of those very assumptions. This thesis shows a tendency to neglect to deal with a long list of partial truths which lead to the whole truth in "the present state of the art" of concrete and reinforced concrete, and its authors missed an opportunity to find clear evidence, in the experiments, to account for the anomaly the Committee devised when "the ratio of the

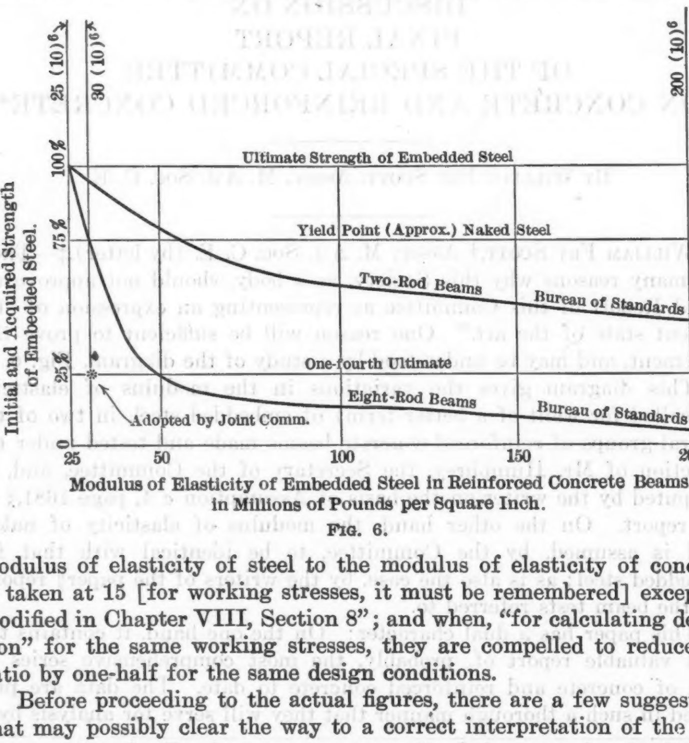


FIG. 6.

modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15 [for working stresses, it must be remembered] except as modified in Chapter VIII, Section 8"; and when, "for calculating deflection" for the same working stresses, they are compelled to reduce the ratio by one-half for the same design conditions.

Before proceeding to the actual figures, there are a few suggestions that may possibly clear the way to a correct interpretation of the data to be reviewed. For instance; on page 43 of the paper by the Bureau of Standards, it is stated, on the thesis side:

"Column 12, Tables 6 to 21, contains the unit elongation of the reinforcement at maximum load. This was obtained by dividing the average total deformation of the reinforcement by the gauge length, 29.25 inches."

Now, the gauge length here referred to is that for the reinforced concrete beam under analysis. Then, again, on page 45 of the same paper, there is the statement: Mr. Scott.

"Given the unit elongation of the reinforcement and assuming a coefficient of elasticity of 30 000 000 the unit stress is easily obtained."

Again, it should be noted that the unit elongation here referred to is that given by the deformations in the plane of the steel in the beam to be analyzed, but the assumed coefficient of 30 000 000 can be none other than that for naked steel.

Immediately following the foregoing quotation the paper states:

"It must be borne in mind, however, that the proportionality of stress and strain holds only within the elastic limit of the material."

Now, a close analysis of the following single example, given by the writer, shows clearly that the modulus of elasticity (so-called) of embedded steel has no relation to that for naked steel, and it also shows that the elastic limit of naked steel bears no relation to that for embedded steel.

Herewith is the example, taken from the paper:

It deals with the gravel concrete beams Nos. 342, 343, and 344, one year old. Twenty-one collateral specimens of the concrete used in these beams were tested in cylinders, 8 in. in diameter and 16 in. long, and developed an average unit load of 5 245 lb. per sq. in. at failure—maximum plus 10% and minimum minus 14½ per cent. The same specimens developed an average modulus of elasticity of 5.395 $(10)^6$ lb. per sq. in. with a maximum of 5.86 $(10)^6$ and a minimum value of 4.8 $(10)^6$ lb. per sq. in.

Now, as to the three beams:

| | Beam No. 342. | Beam No. 343. | Beam No. 344. |
|---|---|---|---|
| Loads given in paper in "analysis of beam at Unit Elongation in Reinforcement Less than that at Yield Point"........ | 6 000 | 5 500 | 6 500 |
| Bending moments, in inch-pounds divided by ($bd^2$)... | 204.5 | 189.5 | 219.6 |
| Deformations, upper fiber, in millionths of an inch per inch. | 325 | 282 | 420 |
| Deformations, steel fiber, in millionths of an inch per inch ..................... | 872 | 603 | 1 111 |
| Neutral axis, distances from top fiber ..................... | 0.271$d$ | 0.319$d$ | 0.275$d$ |

Now take these data and assume, tentatively, that there is straight-line deformation.

Mr. Scott.

| | Beam No. 342. | Beam No. 343. | Beam No. 344. |
|---|---|---|---|
| Then the stresses on the extreme upper fiber of the concrete, in pounds per square inch, would be..... | 1 640 | 1 320 | 1 740 |
| And the moduli of elasticity, on the basis of these stresses and the foregoing deformations, would be........... | 5.05(10)$^6$ | 4.68(10)$^6$ | 4.15(10)$^6$ |

Thus, since the initial modulus of elasticity found for this same concrete ranged from 5.86 (10)$^6$ to 4.8 (10)$^6$ in standard cylindrical specimens, it must be apparent that straight-line stress was practically obtained on these three beams. In fact, comparing Beam No. 342 with its particular collateral specimen, the beam developed 5.05 (10)$^6$ for a stress of 1 640 lb. per sq. in., or 31½% (ultimate) for cylinders, and the cylinder 5.22 (10)$^6$ at initial stress.

Now, this being the status, for the concrete in compression, it is at once apparent that in the three beams there were stresses of 46 000, 43 300, and 49 500 lb. per sq. in. in the reinforcement, if Assumption *c*4 of the report is correct; and then the modulus of elasticity of the embedded steel should equal this unit stress divided by the unit deformation recorded in the "log" for the particular condition of loading under consideration, and would be.... 52.6(10)$^6$ 71.2(10)$^6$ 44.5(10)$^6$ and not 30 (10)$^6$, as laid down by the Committee.

| | | | |
|---|---|---|---|
| Then, again (and this is particularly interesting), the breaking loads on these beams, in pounds, would be. | 6 280 | 5 800 | 6 800 |
| and these exceed that at less than yield point (so-called in the paper) by................. | 5% only | 6% only | 5% only |

In other words, it is found that the modulus of elasticity of the embedded steel in these beams ranges from 71 (10)$^6$ to 44½ (10)$^6$ when they are within 6% of their ultimate strength.

| | | | |
|---|---|---|---|
| Then, again, take the average reading for the three beams at the following applied loads (in pounds)......... | 2 000 | 4 000 | 5 500 |
| Deformations, upper fiber, in millionths of an inch per inch...................... | 76 | 146 | 293 |
| Deformations, steel fiber, in millionths of an inch per inch..................... | 67 | 154 | 650 |

Mr. Scott.

| | Beam No. 342. | Beam No. 343. | Beam No. 344. |
|---|---|---|---|
| Neutral axis, distances from top fiber..................... | $0.53d$ | $0.485d$ | $0.310d$ |
| Bending moments, in inch-pounds divided by $bd^2$...... | 84.4 | 144.4 | 189.4 |
| From which the stresses on the upper fiber, in pounds per square inch, are........... | 387 | 710 | 1 360 |
| and these are the following percentages of the average ultimate unit load per square inch on the twenty-one collateral specimens.......... | 7.4% | 13.6% | 26% |
| for which the moduli of elasticity are................ | $5.1(10)^6$ | $4.85(10)^6$ | $4.65(10)^6$ |
| On the basis of Assumption *c*4 of the report, the stresses on the steel, in pounds per square inch, are................. | 20 700 | 35 200 | 43 000 |
| and these stresses divided by the unit deformation (modulus of elasticity) are.......... | $310(10)^6$ | $230(10)^6$ | $66(10)^6$ |
| and, as 6 290 lb. is the mean maximum applied load for the three beams, it is found that for loads at the following percentages of maximum..................... | 32% | 64% | 87% |
| the moduli of elasticity of embedded steel are........... | 10 times | 7 times | 2 times |
| that laid down by the Committee. | | | |

The concrete of these beams was very rich in cement, but exactly similar results can be shown with concrete beams which are poor in cement.

The writer has developed his own figures from the data in the "Log" of the paper mentioned, and they produce entirely different results from those given in the "Summary of Tests", both in the Bureau of Standards paper.

The two curves on Fig. 6 are based on a general analysis of two groups only, namely two-rod beams and eight-rod beams.

| | | | |
|---|---|---|---|
| Neutral axis distance from top fiber ............ | [illegible] | [illegible] | [illegible] |
| Bending moments, in inch-pounds divided by 10⁶ ...... | 1804 | 2444 | [illegible] |
| From which the stresses in the upper fiber, in pounds per square inch, are ............ | [illegible] | 770 | 980 |
| and these are the following percentages of the average ultimate unit load per square inch on the twenty-one cylindrical specimens ............ | 20.2 | 13.0 | 7.5 |
| for which the moduli of elasticity are ............ | [illegible] | [illegible] | [illegible] |
| On the basis of Assumption 4 of the report, the stresses on the steel, in pounds per square inch, are ............ | 48 000 | 63 200 | 29 700 |
| and these stresses divided by the unit deformation (modulus of elasticity) are ............ | $66(10)^6$ | $290(10)^6$ | $310(10)^6$ |
| and, as 6 200 lb. is the mean maximum applied load for the three beams, it is found that for loads at the following percentages of maximum ............ | [illegible] | [illegible] | [illegible] |
| the moduli of elasticity of embedded steel are ............ | 2 times | 7 times | 10 times |

that laid down by the Committee.

The concrete of these beams was very rich in cement, but exactly [illegible] which are given in [illegible].

The writer has developed his own figures from the data in the [illegible] of the paper [illegible], and these produce entirely different results from those given in the "Summary of Tables" both in the Bureau of Standards paper.

The two curves on Fig. 4 are based on a general analysis of two groups only, namely two-rod beams and eight-rod beams.

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# PAPERS AND DISCUSSIONS



## DISCUSSION ON REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES*

By C. E. Grunsky, M. Am. Soc. C. E.

C. E. Grunsky,† M. Am. Soc. C. E. (by letter).‡—No one who has attempted a solution of the valuation problem as it is presented when the rates of public utilities are to be fixed, will underrate the amount of work done and the careful consideration which has been given by the Committee to the preparation of this report. It represents an earnest effort, in the light of the present state of the art, to lay the foundation for the procedure which should be followed when rates are to be fixed, on the assumption that the decisions of the Courts thus far rendered and as ordinarily interpreted are to be accepted as final. The Committee, however, has failed to recognize the fact that it is unwise to make "value", as called for by the Courts, which results from the earnings, the starting point when rates and, therefore, earnings are to be fixed. The Committee has accepted as reasonable the proposition that there need be no stability to the rate-base, that the rate-base may change with the changing value of the elements of which any public utility plant is composed, and that the determination of this fluctuating, constantly changing, "fair value" is necessary, because the Courts hold that "fair value" is the proper starting point. Mr. Grunsky.

The writer has written so much on this subject, and his views are so well known, that he will make his remarks very brief. He feels certain, however, that the method of procedure when rates are to be

* Continued from February, 1917, *Proceedings*.

† San Francisco, Cal.

‡ Received by the Secretary, February 27th, 1917.

Mr. Grunsky. fixed under which the life of the utility, considered in its entirety, is regarded as unlimited, and under which the use of a definitely ascertainable rate-base (not controlled by value) is possible, has so much in its favor that it will, in time, be generally adopted. This method is not mentioned by name, but is closely related to the method described in the report as the "Replacement Method." The Unlimited-Life Method is, to be sure, a replacement method, but not such as would require the exact amount necessary in any year for replacements to be collected from the rate-payers in that year. Neither is its application limited, as suggested by the Committee, for the Replacement Method. The replacement requirement under the Unlimited-Life Method is to be forecast in a reasonable way, and suitable provision is to be made in the earnings to have the money for making replacements available when required. It is immaterial how the estimate is made; but it is essential that all funds collected for replacement purposes be properly accounted for. If the computation has been at fault, and the replacement fund is found after a time to be inadequate, earnings must be increased. If, on the other hand, there is inordinate accumulation in the replacement fund, then the allowance for the replacement requirement should be reduced. Under the Unlimited-Life Method of procedure the need of estimating the accrued depreciation of the entire plant falls away. The rate-base, once determined, remains unchanged until the plant is extended or modified, or until the public desires to put itself into the position of part owner by retiring a part of the owner's invested capital.

The soundness of the theory on which the Unlimited-Life Method of procedure is based seems to have been generally recognized. The simplicity of its application and the advantage which it has of requiring lowest earnings in the early years of operation, will compel its general adoption or re-adoption whenever the Courts shall have found it desirable to modify their recent decisions which heretofore have been interpreted to hold that value must be made the starting point, ignoring, apparently, the fact, to which attention cannot be called too frequently, that value is not the premise for the determination of earnings, but is created by the earnings.

There is nothing which would prevent the Courts from receding from the position apparently taken in the Knoxville case, in which the United States Supreme Court said:

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends, over issues of securities, or of omission to exact proper prices for the output, the fault is its own."

This position, if it means rates at all times to yield earnings which will include so-called depreciation, is not sound doctrine, because

the cases will be rare indeed in which immediately on the beginning of operations, the earnings can be made adequate to keep the investment unimpaired. Some time must be allowed during which the earnings will fall short of the amount apparently called for by the decision of the Court. This time may be only 1 year, or it may be 5 or 10 years. If the opinion of the Court is regarded as applying with equal force to the years already past as to those yet to come, which seems to follow from the context in the decision, then either a sacrifice may be demanded of the owner, or the few rate-payers of the early years may be required to carry an unwarranted burden. Mr. Grunsky.

It is sound doctrine, and reasonable and fair to both the rate-payer and the owner, to estimate the required earnings so that there will be no immediate retirement of invested capital, but only suitable gradually increasing provision for necessary renewals. The renewal or replacement fund, however, can be regarded at any time as a return of capital, when the business is to be closed out, or when the public desires to participate in the ownership. A method of procedure is thus marked out, which is remarkably simple and makes unnecessary a consideration of value as the starting point when rates are to be fixed.

The rate-base thereunder will start with the capital reasonably and properly invested, not swelled by appreciation nor reduced by depreciation. This is substantially what Mr. James T. Shaw contends for in a statement presented on November 9th, 1915, to the Railroad Commission of California in the Telephone Rate Case (Application No. 1870), when he asks that the "actual performance" be made the "rate-base." In his argument, he reinforces the plea, which is being made by many engineers and economists, that value be regarded as a "result and not a premise."

Influenced perhaps by an earlier presentation of this subject by Mr. Shaw, the Public Service Commission of the State of Washington, in the proceedings relating to telephone and telegraph rates (Cases Nos. 1810 and 1825), on April 25th, 1916, says:

"The statute is silent upon the question of the finding of 'fair value' or a base for rates. The Commission is directed to find the 'market value', but no one contends that the 'market value' is always a fair basis for rates. Since the Commission is required to ascertain the fair, just, reasonable and sufficient rates for telephone service, the Commission will assume that it is authorized to find a 'rate base.'"

Relating to value as a basis for rate-making, the Commission says:

"If we take this definition of the term 'value' and make such value the basis for rate-making, each time we increase the return we increase the desirableness of the property or properties, and on the other hand if we decrease the return, we decrease that which makes the thing desirable; and so, if we decrease the return we decrease the value, and if we increase the return we increase the value."

* * * * * * *

Mr. Grunsky.

"To say that rates are to be based upon the value of the property, using the term in its usual and ordinary sense, is to say that rates shall be based upon one premise to-day, another to-morrow. So we must conclude that when the Courts said that rates were to be based upon 'fair value', they could not have meant to use the word 'value' in the sense in which the word is ordinarily used and understood."

Some Courts, too, are beginning to recognize the futility of requiring that "fair value" shall be made the "basis of the calculation." Thus, for example, the Supreme Court of Idaho in a recent decision (1915), Pocatella Water Company case, Murray *vs.* Public Utilities Commission, reversing the Public Utilities Commission of that State, says in reference to depreciation:

"In other words, if it be demonstrated that the plant is in good operating condition and giving as good service as a new plant, then the question of depreciation may be entirely disregarded."

It is the duty of the valuation engineer and the economist to point out, as occasion arises, that it would be error on the part of the Courts to insist that "fair value" be made the basis of the calculation when rates are to be fixed. It may not be easy, however, to secure general admission that other procedures than the one thus far approved are sound. The Courts must act with caution, and must be convinced beyond peradventure before taking a new stand. The writer believes, however, that the decisions which seem to determine what should be made the "rate-base" are not so final or explicit as to be removed from attack, or at least from possible new interpretation. The decisions of the Courts which make value the starting point, have been rendered to prevent confiscation of property and to protect rate-payers from inordinate exactions. The method supposed to be the only one acceptable to the Courts is illogical and difficult, and is practically impossible of satisfactory application, and yet there is available a simple, logical, and perfectly fair method of procedure, the advantages of which will be generally admitted whenever its application shall have obtained the sanction of the Courts.

Heretofore, the writer has called the attention of the Profession to the fact that the ordinary application of sinking-fund methods to compute the amounts which should go into a depreciation, or into a replacement, fund are materially in error.* It is self-evident that the actual term of usefulness of any article will never agree exactly with the term which was predicted for it when it went into use on the basis of probable life for articles of its class. When consideration is given to the departure of the actual term of service of individual articles from their probable life or expectancy, it will be found that the computation of the annual depreciation increment or of the annual

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 837; also, Vol. LXXIX, p. 761.

Mr. Grunsky.

replacement requirement is quite different from that computed in the ordinary way from their probable life.

It is not proposed to present conclusions on this subject, but only to express regret that the Committee did not take the time to ascertain, approximately at least, the magnitude of the effect of the departure of the actual useful life of articles from their expectancies.* The report is silent on this subject, and, being silent, only reflects the present unsatisfactory state of the art. Any one attempting to apply any method of procedure which involves a deduction of depreciation will find difficulty in making provision in the accounting for the individualized articles which go out of use before the end of their predicted term of usefulness, as well as for those which serve beyond this originally allotted term of usefulness. He will find that the Compound-Interest Method is not, as was originally claimed, an equal-annual-payment method, even when interest rates are uniform, that the Sinking-Fund Method might as well be discarded for an approximation method similar to the Straight-Line Method, and that the Straight-Line Method imposes an unwarranted burden on the rate-payers of the early years, which is undesirable and unnecessary. Any "present-value" method, as already stated, must be accompanied by frequent re-determinations of value. Under all the methods described by the Committee, except the Replacement Method, bookkeeping will be a complicated art.

The application of the Compound-Interest Method of procedure is by no means as simple as the Committee assumes when, on page 1872,† it says:

"There is no ambiguity in the foregoing described method; no chance for confusion; the method is exactly analogous to that described under the straight-line theory of depreciation."

This would only then be true if all individualized articles would actually go out of use at the ends of their respective probable life terms.

The Committee has not offered a satisfactory solution of a vexed question. It leaves the whole matter in confusion. Nevertheless, the facts presented in the report and the academic discussion of the results that would be obtained under various methods of procedure, on the assumption of certain premises, not to be realized in practice, is a valuable contribution to the literature on valuation and rate regulation, and will be of material aid in hastening the acceptance of fundamental principles and the general adoption of a satisfactory method of procedure.

* See "Valuation, Depreciation, and the Rate-Base", by C. E. Grunsky, pp. 104 to 122.

† *Proceedings*, Am. Soc. C. E., December, 1916.

replacement requirement is quite different from that computed by the ordinary text book three-percent line.

It is not proposed to discuss exhaustively this subject, but only to express regret that the Committee did not take the time to work, being approximately, at least, the magnitude of the effect of the diversity of life which would tend to average from their expectancies.* The writer is aware of the subject, and being silent, only reflects the present unsatisfactory state of the art. The writer anticipates that any method of procedure which involves a presumption of depreciation will find difficulty in meeting the claim of the accountants that the individualized articles equal service of the items the end of their generation term, or reductions as writing for those which serve beyond this reasonably allotted term of usefulness. He will find that the Compound-Interest Method is not, as some claim, condemned as simple, since the Sinking Fund Method would as well be determined for accompanying reduction similar to the Straight-Line Method, and that the Straight-Line Method imposes an unwarranted burden on the investors of the early years, which is unjust and unnecessary. Any "present-value" method as already stated, must be accompanied by frequent re-determinations of value. Under all the methods described by the Committee, except the Straight-Line Method, bookkeeping will be a complicated art.

The application of the Compound-Interest Method of procedure is by no means as simple as the Committee assumes when, on page 1874,† it says:

"There is no ambiguity in the foregoing described methods; no chance for confusion; the method is exactly analogous to that described under the straight-line theory of depreciation."

This would only then be true if all individualized articles would actually go out of use at the ends of their respective probable life terms.

The Committee has not offered a satisfactory solution of a vexed question. It leaves the whole matter in confusion. Nevertheless, the facts presented in the report and the academic discussion of the results that would be obtained under certain methods of procedure, on the assumption of certain premises, not to be realized in practice, is a valuable contribution to the literature on valuation and rate regulation, and will be of material aid in hastening the acceptance of fundamental principles and the general adoption of a satisfactory method of procedure.

* See "Valuation, Depreciation, and the Rate-Base", by C. E. Grunsky, pp. 104-

† Proceedings, Am. Soc. C. E., December, 1916.

# MEMOIRS OF DECEASED MEMBERS

**NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.**

## THOMAS APPLETON, M. Am. Soc. C. E.*

DIED AUGUST 3D, 1916.

Thomas Appleton, the son of Edward and Frances Anne Atkinson Appleton, was born at Reading, Mass., on October 13th, 1846. He was of Colonial ancestry, his father's family having settled in Ipswich, Mass., in 1635. His mother was a descendant of Simon Bradstreet, an early Colonial Governor of Massachusetts and New Hampshire, and Anne Dudley.

Mr. Appleton's father, Edward Appleton, was a Civil Engineer, having been graduated from Harvard College in the Class of 1835, and a member of the first Board of Railroad Commissioners of Massachusetts. In 1862, after completing his studies in the public schools of Reading, Thomas Appleton entered the employ of the Boston, Hartford and Erie Railroad as Rodman, continuing with this Company until he enlisted, for the Civil War, in Company E, Eighth Massachusetts Volunteer Infantry, from which he was honorably discharged in November, 1864.

In 1865, Mr. Appleton entered Rensselaer Polytechnic Institute, from which he was graduated with the Class of 1868 with the degree of Civil Engineer. Among his classmates at Rensselaer were M. T. Endicott, Past-President, Am. Soc. C. E., and the late Virgil G. Bogue, L. L. Buck, and O. F. Nichols, Members, Am. Soc. C. E.

After his graduation, Mr. Appleton was appointed Resident Engineer with the Dexter and Newport Railroad Company, on construction work in Maine, and, in 1869, he was made Division Engineer on the Somerset Railroad. In 1870, he was appointed Engineer in charge of the reconstruction of the Troy and Greenfield Railroad (now a part of the Fitchburg Division of the Boston and Maine System) east of the Hoosac Tunnel, in Massachusetts. Recently, the old steel work on this line was replaced to accommodate the heavier loads and train movements of these days, but the abutments of the two bridges across the Deerfield River, constructed under Mr. Appleton's direction, were not disturbed or strengthened.

On the completion of this work, in 1871, Mr. Appleton was appointed Chief Engineer of the Essex Branch and Marginal Freight

* Memoir prepared by the Secretary from information on file at the House of the Society.

Railroads in Massachusetts, and, in 1872, he was made Principal Assistant Engineer of the Boston and Maine Railroad.

From 1873 to 1879, he was engaged on surveys for several railroads, and acted as Inspector of Bridges for the City of Boston and also as Traveling Agent for the Niagara Bridge Works, of Buffalo, N. Y.

In 1880, Mr. Appleton was made Superintendent of Bridges and Buildings for the Galveston, Harrisburg and San Antonio Railway (now a part of the Southern Pacific System in Texas) and, in 1881, he was engaged in making location surveys in New York and Connecticut for the New York, Boston, Albany and Schenectady Railroad. He left the latter work to become Chief Engineer of the Kansas and Eastern Construction Company and, in this capacity, built the Leavenworth, Topeka and Southwestern Railroad. In 1883, he was employed as Assistant Engineer on the Topeka, Salina and Western Railroad.

In 1884, Mr. Appleton was appointed City Engineer of East Saginaw, Mich., but preferring railroad work, he resigned to accept the position of Principal Assistant Engineer on the Chicago, Milwaukee and St. Paul Railroad, which he held from 1885 to 1889. He then went to the Union Pacific Railroad where he served as Engineer of Buildings until the latter part of 1890. The late Mr. Bogue, one of his classmates at college, was Chief Engineer of the Union Pacific Railroad at that time, and on hearing of Mr. Appleton's death, he wrote of him as follows: "I always had a great affection for him and always found him a sweet-tempered, kindly-disposed man, who did his duty."

In 1891, Mr. Appleton opened an office in Chicago, Ill., for the private practice of engineering, which he continued until 1895, when he was appointed Engineer of Buildings and Water Supply with the Great Northern Railroad. He resigned this position in 1898 to accept that of Chief Engineer of the Copper Range Railroad.

In 1890, Mr. Appleton entered the Government service as Assistant to the Superintendent of Construction of the United States Life Saving Service, in which capacity he had charge of the construction of many of the life-saving stations along the Atlantic Coast and the Great Lakes.

In 1902, at his own request, Mr. Appleton was transferred from the Life Saving Service to the office of the Supervising Architect, Treasury Department, as Superintendent of Construction, United States Public Buildings, which position he held until his death at Gardiner, Me., on August 3d, 1916. His first work while holding this office was on the construction of the Post Office Building at Creston, Iowa. This was followed by work on public buildings at Holyoke and Northampton, Mass., Wheeling, W. Va., Evanston, Wyo., East St.

Louis, Alton, Centralia, and Greenville, Ill., Chelsea, Mass., the new Custom House at Boston, Mass., New Bedford, Mass., and Gardiner, Me. After the earthquake and fire in San Francisco, he was ordered there to assist in the work of temporary repairs to protect Government buildings. The deep and difficult foundations for the new Custom House at Boston, Mass., were constructed under his supervision, but at his request and on account of advancing years, he was relieved from that detail and transferred to New Bedford in 1913.

Mr. Appleton was married to Mary Louise Burnham, of Essex, Mass., on October 1st, 1871, and is survived by his widow, one brother, four sisters, and two stepsons.

He was the founder and first President of the Society of Constructors of Federal Buildings, which was organized, principally through his efforts, in 1910, with the idea of promoting closer relations between the Office and Field forces of the Supervising Architect's Office, and thereby raising the efficiency of both. The success of this organization, which has grown from a few members to include not only the entire Field force of the Office, but also a considerable number of the Office force, is due to Mr. Appleton's never-failing interest and activity in its affairs. He attended all its Annual Conventions and was a generous contributor to the pages of its *Journal,* his last paper, on the history and origin of the Society, having been published in May, 1916.

In 1874, Mr. Appleton joined the Boston Society of Civil Engineers and continued his membership in that organization for many years after going West. He was a charter member of the American Engineering and Maintenance of Way Association (now the American Railway Engineering Association), and while in Chicago, he joined the Western Society of Engineers and served as its Secretary for a year or more. He was also a member of the American Society of Mechanical Engineers from 1893 to 1910, a member of Kilpatrick Post No. 71, G. A. R., of Holyoke, Mass., and of the Zeta Psi Fraternity.

The following extract from a letter from the Acting Supervising Architect, Mr. James A. Wetmore, to the Secretary of the Society of Constructors of Federal Buildings, expresses the high estimate in which Mr. Appleton was held by all who knew him:

"* * * In every relation with him, official and personal, I was impressed with his devotion to duty, his ideals of rectitude and honor, his kindly disposition, and his consideration for others. Endowed with a pleasing personality, possessed of the requisite technical qualifications, gifted with tact and a disposition to find a way to overcome difficulties, it was natural that he should be highly regarded by his associates and co-workers in the Office and in the Field. I feel that in his death, the public service has lost a faithful servant, your Society a devoted adherent, and all of us a sincere friend."

Mr. Appleton's immediate superior, Mr. George O. Von Nerta, Technical Officer of the Supervising Architect's Office, writes:

"* * * Where he stood out apart from the rest was in his ability to conceive and carry into execution an improvement of far-reaching effect."

Mr. Appleton was elected a Member of the American Society of Civil Engineers on April 4th, 1883.

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## JOHN WALDO ELLIS, M. Am. Soc. C. E.*

Died October 30th, 1916.

John Waldo Ellis, son of John and Ame Almira Fisher Ellis, was born in Woonsocket, R. I., on September 7th, 1845. He was one of four children, of which three were boys, all noted civil engineers.

Mr. Ellis received his early education at New Hampton Institute, New Hampton, N. H., and at the age of 19 entered the employ of the Boston, Hartford and Erie Railroad, then building from Waterbury, Conn., to Fishskill, N. Y. His advancement was rapid, and he soon became a Division Engineer on the Troy and Greenfield Railroad before the completion of the Hoosac Tunnel. From this road he went to the Norwich and Worcester Railroad then building, and, in 1869, came to Woonsocket, R. I., as Chief Engineer of the Providence and Worcester Railroad, at the same time opening an office for private practice in that place, which he maintained up to within a few years of his death.

Mr. Ellis held the position of Chief Engineer of the Providence and Worcester Railroad, up to the time that road was absorbed by the New York, Providence and Boston Railroad in 1888. Under his direction the road was double-tracked, many branch lines were constructed, the Wilkes-Barre Coal Pier and connection was constructed at Providence, and many bridges, stations, and terminals were rebuilt. During this same period Mr. Ellis' private practice in Woonsocket was at its height, and many prominent engineers of the present day received their first experience in the old Main Street office. The design and direction of the construction of Nourse Mill, of the Social Manufacturing Company, the Alice Mill, of the Woonsocket Rubber Company, and numerous other industrial plants and enlargements in Northern Rhode Island, were a part of the activities of this office.

From 1890 to the time of his death, Mr. Ellis was connected prominently with the various engineering problems of the East. He was Engineer for the Old Colony Railroad Company in the building of the Providence Passenger Terminal, and Engineer Inspector of the

* Memoir prepared by Lester Waldo Tucker, M. Am. Soc. C. E.

Boston and Providence Railroad, from the time of its lease to the Old Colony Railroad, until his death. His connection with various grade crossing matters in Massachusetts and Rhode Island included nearly every important problem that has come up. He was one of the Commissioners for the abolishment of the grade crossings in Lowell, Athol, and Orange, Mass., and was employed as engineering expert by the Cities of Lynn, Worcester, Cambridge, Fall River, Taunton, Haverhill, Readville, and a large number of small towns.

As a Water-Works Expert, Mr. Ellis was among the foremost in New England, serving as one of the Commissioners in the valuation of the Newburyport and Gloucester Water-Works when these were taken over by the City. He was also a member of the Commission in the diversion claims against the City of Pittsfield and the claim of the Nassau Paper Company against the Metropolitan Water Board.

As a Town and City Engineer, Mr. Ellis was especially active, serving as Town Engineer of Woonsocket from 1870 to the time it became a city in 1888. He also served as Engineer for the Town of Blackstone, and for other surrounding towns, up to the time of the closing out of his private practice.

As a Hydraulic Engineer, Mr. Ellis was very active, and the Blackstone and other rivers in Massachusetts and Rhode Island have many a dam constructed under his direction. The most prominent of these are the Lonsdale, Ashton, and Wilkinsonville Dams on the Blackstone; the Slatersville Reservoir Dam and Middle Dam on the Branch River, and the Georgiaville Dam on the Woonasquatucket, in Rhode Island.

On March 1st, 1901, Mr. Ellis was elected President of the Providence Gas Company, and took up the active management of that Corporation, holding the position of President and General Manager to the time of his death. Under his direction, this Company became one of the most efficient of the gas companies operating in the East.

Notwithstanding his many engineering engagements and business connections, Mr. Ellis found time to be a most efficient Director and Manager in other fields. He was a member of the Board of Directors of the Industrial Trust Company, of Providence, and Chairman of the Board of the Woonsocket Branch of that Company. He was a Director in the Woonsocket Rubber Company, and many other Corporations. He was also a Trustee of the Woonsocket Institution for Savings, from 1876 to 1908, and a Trustee of the Woonsocket Hospital from its founding, in 1890, to the time of his death. Although a prominent member of many clubs and social organizations, Mr. Ellis had no connection with any fraternal or secret orders. He was a member of the Boston Society of Civil Engineers, serving as President of that Society in 1905. He was also a member of the New England Water Works Association.

His principal diversion in his leisure was that of driving. From the time when he established his home in Woonsocket his stable always contained at least one good blooded trotting horse, and when the roads were good or the sleighing at its best, Mr. Ellis was to be seen among the fastest of those on the speedways. He was a member of the Woonsocket Driving Club, the Roger Williams Driving Club, of Providence, and the Metropolitan Driving Club, of Boston, and it is interesting to note that only three weeks before his death he drove on the track of the latter Club.

Mr. Ellis was a prominent figure in the political field of Woonsocket for many years. He served as Alderman from his Ward from 1895 to 1897 and was President of the Board during the last two years of this service. In 1904, he was elected State Senator from his city and served on many important Committees.

He was a member of the Board of Trustees of the First Universalist Church for many years.

Mr. Ellis was a man of such marked ability in any of the fields into which he entered that he was recognized as an authority on an unusual range of engineering problems, a public man and a statesman of great ability, and a business man with keen foresight and tremendous energy. The scope of his talents was wide, and indicated a breadth of mentality seldom found in one man.

Mr. Ellis was married on May 23d, 1870, to Mary F. Howe, who, with one son and two daughters, survives him.

He was elected a Member of the American Society of Civil Engineers on July 3d, 1895, and served as a Director from 1904 to 1906, inclusive.

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## DANIEL McCOOL, M. Am. Soc. C. E.*

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Died November 30th, 1916.

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Daniel McCool was born at London, Ont., Canada, on January 9th, 1850. He was educated at the Jesuit College in Quebec, and, later, was graduated from the English High School in that city.

In 1869, Mr. McCool moved to Niagara Falls, N. Y., and later to Auburn, N. Y., and his first engineering experience was gained as Rodman on the fortifications at Point Lewis where he was employed for two years.

He then entered the employ of the New York and Oswego Midland Railroad Company as Assistant Engineer on construction. In 1873, he

* Memoir prepared by the Secretary from information on file at the House of the Society.

became Assistant Resident Engineer on the New York Central and Hudson River Railroad, in which position he had charge of the four-track improvement west of Syracuse, N. Y., under the Chief Engineer. During the last five years of his service with this Company, Mr. McCool, in addition to his engineering duties, served as Assistant Superintendent in charge of traffic.

In 1880, Mr. McCool was appointed Private Secretary to the General Manager of the Michigan Central Railroad and also had charge of the Engineering Department of the Company. While in this position, he designed and constructed the new freight yards and stockyards at Detroit and made the plans for the new Passenger Station at that place, as well as the improvements connected therewith.

In 1882, he entered the service of the Detroit, Mackinaw and Marquette Railroad Company as General Superintendent and Chief Engineer. In this capacity he had charge of the car ferry service over the Straits of Mackinac, and introduced the first successful ice-crushing craft, by placing a propeller at the bow as well as at the stern of the ferry. While in the employ of this Company he extended the line to Ishpeming and Negaunee, Mich., in order to tap the extensive iron mines in that region.

In 1885, Mr. McCool was made General Manager of the St. Joseph and Grand Island Railroad, with headquarters at St. Joseph, Mo. He was also President of the St. Joseph Terminal Company, and built the Kansas City and Omaha Railroad, a distance of 200 miles, as a feeder to the St. Joseph and Grand Island.

In 1888, he became General Manager of the Santa Fé and California Railroad, and during his connection with that Company added 100 miles to its lines.

In 1889, Mr. McCool retired from railroad work and made an extended tour in Europe. On his return to the United States, he organized, in 1898, the Newaygo Portland Cement Company, at Newaygo, Mich., with which he was identified at the time of his death, having served as its President for many years and having been largely instrumental in putting it on a paying basis. Under his direction, the marl beds near Newaygo, belonging to the Company, were developed, the local power dam was built, and the great plant of the Company was constructed. In addition to his work as President of the Newaygo Portland Cement Company, Mr. McCool was prominently identified with the Edison Electric Light Company, of Grand Rapids, Mich., of which he became President in 1901. He was also interested in many other local enterprises.

In the summer of 1899, Mr. McCool sustained serious injuries in a runaway accident from which he never fully recovered. A short time before his death, he had entered the Henry Ford Hos-

pital, at Detroit, Mich., for treatment, and had returned to his home in Grand Rapids apparently greatly improved. He had continued actively in the business of the Newaygo Portland Cement Company and had visited the plant only a few days before the brief illness which caused his death.

Mr. McCool is survived by his widow, who was Miss Kate Fisher, of Batavia, N. Y., and one brother, Mr. William A. Tench, of Detroit, Mich.

In Grand Rapids and Newaygo, Mr. McCool was widely known for his benevolent and philanthropic work, having been actively identified with many charitable movements in both places. In the business world his influence was strong and progressive, and he commanded high esteem among men of large affairs. His funeral took place at Batavia, N. Y., and out of respect for him who had been its founder and President, the hour was marked by the complete cessation of operations at the plant and power house of the Newaygo Portland Cement Company.

Mr. McCool was elected a Member of the American Society of Civil Engineers on September 5th, 1883.

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## THOMAS FRANKLIN RICHARDSON, M. Am. Soc. C. E.*

DIED DECEMBER 26TH, 1915.

Thomas Franklin Richardson was born at Woburn, Mass., on February 10th, 1855. He was of Colonial ancestry, having been descended from Samuel Richardson who came to this country about 1630 from the south of England and settled in what is now Winchester, Mass.

Mr. Richardson was graduated in 1872 from the Boston English High School, and began his career as a Civil Engineer in May, 1873, as Rodman on the location and construction of the aqueduct for the Sudbury Water-Works, for the City of Boston. When the work was finished in December, 1878, he had been promoted to the position of Instrumentman.

From December, 1878, to October, 1879, he was employed as Assistant Engineer for the City of Chelsea, Mass., in charge of the construction of the main trunk sewer for that place.

In January, 1880, Mr. Richardson went West as Topographer on surveys for the location of the Atlantic and Pacific Railroad (now part of the Santa Fé System), and in seven months was made Division Engineer, which position he held until June, 1882. As Division

* Memoir prepared by the Secretary from information on file at the House of the Society.

Engineer, he had charge of the construction of about 25 miles of railroad in New Mexico and Arizona and was also active in the management of that portion of the road. His work here included the construction of the Cañon Diablo and the Padre Cañon Bridges in Arizona, the former of which was a notable piece of engineering for that time, being a steel trestle about 225 ft. high and 600 ft. long. Labor conditions were extremely bad in this country, and water for construction purposes had to be hauled from 25 to 30 miles, largely by teams, thus making the work very difficult.

In October, 1882, Mr. Richardson joined the Engineering Corps of the Mexican Central Railway Company as Assistant Engineer. In this capacity he had charge of the construction of the Encarnacion Bridge, 175 ft. high and 1 300 ft. long. At that time this bridge was considered to be the most important piece of bridge work undertaken in Mexico. He remained with this Company until July, 1885, serving for about a year as Bridge Engineer on the southern half of the road and, for another year, as Resident Engineer on the northern 600 miles of the same road, designing and constructing a great many of the bridges.

In September, 1885, Mr. Richardson was appointed Division Engineer on the construction of the Chicago, Burlington and Northern Railroad in Wisconsin, in charge of timber trestle work over the Wisconsin River and also of the heavy grading work. In July, 1886, he entered the employ of the Colorado Fuel Company, making railroad surveys, designing structures, and making estimates for the development of that Company's coal properties in the mountains between Glenwood Springs and Aspen, Colo. He also located a number of railroad lines in this region with grades as high as 5%, using switchbacks on many of the lines. He remained with the Colorado Fuel Company until May, 1887, when he was made Division Engineer on the construction of the Atchison, Topeka and Santa Fé Railroad. In this capacity he had charge of the Division Yard at Chillicothe and the heavy work near the Illinois River. For the greater part of the last year (1888) he served as Resident Engineer in charge of all new work in the State of Illinois.

From January to April, 1889, Mr. Richardson was engaged on the location of a line in the San Luis Valley, from Valley Grove to Alamaso, for the Denver and Rio Grande Railroad, and, from April, 1889, to January, 1892, he was Chief Engineer on the location and construction of the Manitou and Pike's Peak Railroad. This road has a maximum grade of 25 ft. per 100, with a maximum curvature of 16°, is 9 miles long, and extends from an elevation of about 6 000 ft. to one of 14 100 ft. above sea level, about 3½ miles of it being above the timber line. Mr. Richardson designed all the structures and all the appliances for laying the Abt rack rail which was used on this line.

In February, 1892, he went to Chicago, Ill., where he entered the service of the Sanitary District as Assistant Engineer, having charge of the construction of several miles of the Lemont portion of the main channel, and from April to August, 1893, he served as Engineer in charge of the construction of the 68th Street Tunnel (one of the water tunnels built of brick through clay and silt 4 miles under Lake Michigan) for the City of Chicago.

In August, 1893, Mr. Richardson returned to Massachusetts where he entered the service of the State Board of Health and, until June, 1895, had charge of surveys for all the reservoirs and aqueducts in connection with the investigations for a water supply for the Metropolitan District. He made the preliminary locations, designs, and estimates of cost, for the location of the Wachusett Aqueduct; the Wachusett Reservoir and Dam, including all appurtenances thereto; a reservoir on Ware River, together with a tunnel 8.7 miles long connecting it with the Wachusett Reservoir; a reservoir on Swift River, including a tunnel 27.6 miles long, also connecting this reservoir with the Wachusett Reservoir; and the Weston Aqueduct, as well as many other sources of supply.

From July, 1895, to January, 1906, Mr. Richardson was employed as Engineer of the Dam and Aqueduct Department of the Metropolitan Water-Works, in charge of the construction of the Wachusett Reservoir, 12 miles long, which work included a tunnel 2 miles long and a masonry arch bridge over the Valley of the Assabet River. He also had charge of the construction of the sewage filtering system for Clinton, Mass., including a power-house and 25 acres of filter beds. Until May, 1906, he had charge of studies for the location and design of the North Dike and the land surveys for the Wachusett Reservoir. This work included the relocation of the Central Massachusetts Railroad from Berlin Junction to the North Dike, with a steel viaduct about 135 ft. high and 1 000 ft. long, across the Nashua River below the dam. Mr. Richardson also made the final location for the Weston Aqueduct and was in charge of the construction work in connection with the Wachusett Dam, as well as serving as Engineer of the Reservoir Department in 1905.

In January, 1906, Mr. Richardson entered the employ of the J. G. White Company, Incorporated, as Construction Superintendent in charge at the Laguna Dam near Yuma, Ariz., for the United States Reclamation Service.

In March of the same year, he returned to the Metropolitan Water-Works of Boston, Mass., as Engineer of the Dam and Reservoir Department, which position he held until July, when he was appointed Resident Manager in charge of the construction of the plant of the Rockingham Power Company on the Peedee River at Hamlet, N. C., where he remained until July, 1907.

In September, 1907, Mr. Richardson again joined the forces of the J. G. White Company, Incorporated, and was placed in charge of the La Crosse water-power development at Hatfield, Wis. In December, 1907, he returned to New York City, and on January 1st, 1908, was appointed General Superintendent of Construction in charge of contract work for the New York office. In this capacity, he had charge of various large hydro-electric works, such as the La Crosse Power Company's development, at Hatfield, Wis.; the Connecticut River Power Company's development near Brattleboro, Vt.; the Central Georgia Power Company's development at Jackson, Ga.; the Idaho-Oregon Light and Power Company's development at Copperfield, Ore.; the Minnesota-Ontario Power Company's development at International Falls, Minn., as well as the Idaho Irrigation development in the vicinity of Richfield, Idaho. Mr. Richardson continued as General Superintendent of Construction until September 1st, 1909, when he was made Chief Civil Engineer of the Company. While in this position, he made numerous investigations, studies, and cost estimates for power developments and also acted as Consulting Expert in engineering matters for the Company. During this time he was also engaged as Consulting Engineer in connection with the Fellsmere Farms Company's work of reclaiming swamp lands in Florida.

When the Engineering Department of the J. G. White Company, Incorporated, was taken over by its subsidiary, The J. G. White Engineering Corporation, January 1st, 1913, Mr. Richardson became Civil Engineer for the latter, which position he retained until August 1st, 1914, when he was obliged to resign on account of failing health. He continued, however, to engage in consulting work until a short time before his death which occurred at his home at Rutherford, N. J., on December 26th, 1915.

Mr. Richardson was engaged for more than 40 years in all kinds of engineering work, including the construction of railroads, water supply, irrigation, drainage, and hydro-electric projects. His work extended over all parts of the United States, which fact made him widely known as a capable engineer, an energetic executive, and a faithful and honest man.

On October 4th, 1888, Mr. Richardson was married to Miss Nellie Symonds who, with three sons, survives him.

He was a member of the Boston Society of Civil Engineers, the New England Water Works Association, and the Engineers' Club of New York City. He was also a Knight Templar and a Mystic Shriner.

Mr. Richardson was elected a Member of the American Society of Civil Engineers on November 4th, 1885.

## EDMUND BROWNELL WESTON, M. Am. Soc. C. E.*

DIED DECEMBER 9TH, 1916.

Edmund Brownell Weston, the son of the Hon. Gershom Bradford and Deborah (Brownell) Weston, was born in Duxbury, Mass., on March 25th, 1850. He was descended from a family famous as shipbuilders and shipowners whose vessels were known in every seaport of the world. In 1842, the firm, the E. Weston Company, was rated by Lloyd's, of London, as the greatest shipping concern in the world. Mr. Weston's father was a member of the Convention which nominated John C. Fremont for the Presidency in 1856, and also of that which nominated Abraham Lincoln in 1860. His mother belonged to the Brownell family of Little Compton, R. I.

Mr. Weston was educated in the public schools and at the Partridge Academy of his native town, the Highland Military Academy at Worcester, Mass., and by private tutors.

From 1871 to 1874, he was a student in the office of the late Joel Herbert Shedd, M. Am. Soc. C. E., then Chief Engineer of the Providence, R. I., Water-Works which were being constructed at that time. In 1874, Mr. Weston was appointed Assistant Engineer on that work, and held that position until 1877 when Mr. Shedd resigned. Early in 1877, when Samuel M. Gray, M. Am. Soc. C. E., was elected Chief Engineer and Superintendent of the Water-Works to succeed Mr. Shedd, Mr. Weston was appointed one of his Assistant Engineers, and, in this capacity, had charge of the Water Department until 1897, when he resigned to engage in private practice as a Consulting Engineer.

Mr. Weston had been in poor health for a year, but had only been confined to his bed about two months when his death occurred, from hardening of the arteries, at the Des Brisay Hospital, at Boston, Mass. He is survived by two brothers and one sister, all residing in Boston, Mass.

At the time of his death, Mr. Weston was President of the Jewell Export Filter Company, of Providence, R. I., and was widely known as a hydraulic engineer. He had designed and constructed filtration systems, in connection with water-works, in many cities, both in America and abroad, and had traveled practically over the civilized world studying systems of water filtration wherever he went. Until the war broke out in Europe, in 1914, he had spent every summer abroad.

In addition to his practical work on hydraulics, Mr. Weston had contributed much to the literature on the subject, his best known

* Memoir prepared by the Secretary, from information on file at the House of the Society.

work being "Friction of Water in Pipes", published in 1873. He had also contributed many discussions and four papers on the subject to the *Transactions* of the Society, namely, "Description of Some Experiments on the Flow of Water Through 2½-Inch Rubber Hose and Nozzles of Various Forms and Sizes, Made in the Providence, R. I., Water-Works. Also Results of Investigations Relating to the Height of Jets of Water" (1884); "Description of Some Experiments Made in the Providence, R. I., Water-Works, to Ascertain the Force of Water in Pipes" (1885); "The Results of Investigations Relative to Formulas for the Flow of Water in Pipe" (1890); and "Test of a Mechanical Filter" (1900).

Mr. Weston was a member of the University Club, of Providence, R. I.; the India House Club, of New York City; the Boston Society of Civil Engineers; the Institution of Civil Engineers of Great Britain; and the Authors and Royal Societies of London, England. He was also a member of Corinthian Lodge, F. and A. M., of Providence, R. I.

The following resolution on the death of Mr. Weston was adopted unanimously by the Directors of the Jewell Export Filter Company:

*Resolved:* That the Company and the Board of Directors have suffered a great loss in the death of Mr. Edmund B. Weston, so long the President and General Manager of this Company, and its Consulting Engineer, and that the Board take this occasion to express its appreciation of the long and faithful service that Mr. Weston rendered the Company, fully realizing that he at all times gave the Company that valuable service which was the result of his intimate knowledge of the business and affairs of the Company and the matters in which it has been engaged, and further realizing that Mr. Weston rendered this service oftentimes handicapped with the discouragement of physical ailments which would have taxed the determination of men less faithful than he; and it is further

*Resolved:* That copies of the above resolution be sent to the Executors of Mr. Weston's estate.

Mr. Weston was elected a Member of the American Society of Civil Engineers, on December 6th, 1882.

## HOWARD ARNOLD GREENE, Assoc. M. Am. Soc. C. E.*

Died February 2d, 1917.

Howard Arnold Greene was born at Providence, R. I., on September 15th, 1860.

Immediately after his graduation from Brown University, in June, 1884, Mr. Greene entered the employ of S. B. Cushing and Company,

* Memoir prepared by the Secretary from information on file at the House of the Society.

Engineers and Surveyors, and remained with that firm until April, 1887. While with this Company, Mr. Greene was engaged on the construction of the Diamond Hill Reservoir for the Pawtucket, R. I., Water-Works, on the greater part of the line of the Union Street Railroad Company, and, as Assistant Engineer, on all the preliminary work for the Providence Cable Railroad Company and the improvements of the Park Land Company, at Elmwood, R. I.

In April, 1887, Mr. Greene was appointed Division Engineer with the Hudson Connecting Railroad Company, under the late P. P. Dickinson, M. Am. Soc. C. E., as Chief Engineer, on the Poughkeepsie Bridge route. When this was finished, in September, 1888, he accepted a position as Engineer for J. W. Coffin and Company, Contractors for the Brigantine Beach Railroad, which position he retained until August, 1889, when he went to the Dunderbergh Spiral Railroad Company, as Division Engineer, until work on that line was stopped.

In December, 1889, Mr. Greene was appointed Locating Engineer with the Danville and East Tennessee Railroad Company. He remained in that position until May, 1890, when he entered the employ of Morris and Darly, Engineers and Contractors, of Bristol, Tenn., as Assistant Engineer on surveys for the State of Virginia and on the erection of a blast furnace at Bristol, Tenn.

In December, 1890, Mr. Greene was engaged as Engineer in charge of the construction of the terminal pier at Atlantic Highlands, N. J., for the Central Railroad of New Jersey. He afterward went to Connecticut as Assistant Engineer on the location of about 30 miles of railroad, under the late Joseph Norton Greene, M. Am. Soc. C. E., as Chief Engineer.

In November, 1892, Mr. Greene entered the employ of the New Jersey Steel and Iron Company as Assistant Engineer. He was promoted, until at the time of the incorporation of the American Bridge Company, he was in charge of erection in the New York District.

In July, 1901, he was appointed Erection Manager of the Pittsburgh Division for the American Bridge Company, which position he held at the time of his death. While serving in this capacity, Mr. Greene had charge of the erection of many important structures and had invented a number of important erection devices which are patented.

He died of pneumonia at his home in Pittsburgh, Pa., on February 2d, 1917, after a brief illness, and is survived by his widow.

Mr. Greene was elected an Associate Member of the American Society of Civil Engineers, on June 5th, 1895.

## PAPERS IN THIS NUMBER



## PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

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PAPERS AND DISCUSSIONS

APRIL, 1917

Vol. XLIII. APRIL, 1917. No. 4.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



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## AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA

By F. C. Carstarphen, Assoc. M. Am. Soc. C. E.

To be Presented May 2d, 1917.

### Synopsis.

**This paper gives a brief statement of the prominent features of the location, construction, and operation of an aerial tramway built to carry salt, at the rate of 20 tons per hour, a distance of 13½ miles, from Saline Valley, over the Inyo Mountains, to a point in Owens Valley, California.**

**It is assumed that the reader is familiar with the terminology of aerial tramway design; also, that the ability of an aerial tramway system to transport materials economically and efficiently at the rate of 250 tons per hour or less is so well known as to make extensive discussion of these points unnecessary.**

California now possesses one of the most novel aerial tramways ever constructed. This was built over the Inyo Mountains for the Saline Valley Salt Company, of Los Angeles, Cal.

In 1904 Adolf Bleichert and Company completed for the Argentine Republic, a tramway, 22 miles in length, with a difference in terminal

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

elevations of 11 500 ft., thereby establishing a record for length of lines operated continuously and successfully. In 1915 the American Steel and Wire Company completed an aerial tramway for the Spring Canyon Coal Company, of Storrs, Utah, which carries 285 tons of coal per hour.

Although these two structures exceed the Saline Valley tramway in length and capacity, respectively, neither of them crosses a mountain range, nor does either show such boldness of engineering design. The engineers who undertook the construction of this tramway were fitted by experience to solve the many problems which confronted them. As early as 1895 the Trenton Iron Company designed a tramway, 12 miles long, for the Compagnie Haitienne, of Port de Paix, Haiti, and it has been maintained in successful operation ever since.

The purpose of the Saline Valley Salt Company, in building this aerial tramway, was to transport salt from Saline Valley, over the Inyo Mountains, to the railroad in Owens Valley. The tramway is in Inyo County which is remarkable as possessing the highest and lowest spots in the United States. Mt. Whitney, elevation 14 501 ft., is the crowning height of the Sierra Nevada Mountains, forming the west wall of Owens Valley; Death Valley, to the east, has an elevation of minus 280 ft., and is the lowest spot in the desert. An observer standing on the summit of Telescope Peak of the Panamint Range can view the extremes in elevation of the Republic.

It may be remarked in passing that Owens Valley, elevation approximately 3 600 ft., possesses a river of such magnitude that the City of Los Angeles has built an aqueduct, 274 miles long, for the purpose of making this mountain stream available for the domestic water supply of the city. The Owens River flows into Owens Lake, which is a dead sea, without outlet. The waters of this lake are sufficiently rich in sodium carbonate to make its extraction profitable. At present, plants on the east side of the lake are producing soda ash and sodium bicarbonate in large quantities.

To the east, beyond the Inyo Mountains, is Saline Valley, with an elevation of 1 100 ft. This valley has no water system, but is remarkable for several large springs, and there is flowing water in Hunter Canyon. To the east of Saline Valley lies the romantic Death Valley, its depressed floor dotted with borax deposits. Thus, it may be noted, the commercial worth of these valleys depends on altitude. The tempera-

ture of the summer air of Owens Valley is not excessive, but Saline and Death Valleys are no doubt quite the hottest places that can be found in the world. Temperatures of 120° Fahr. in the shade prevail for considerable periods, and only because of the absence of humidity can the summer heat be endured.

The topography of Inyo County owes its extreme boldness to the simple nature of the faulting, which is responsible for the Sierra Nevada and Inyo Mountains. When the great blocks of the earth's crust were thrust upward they assumed a tilt toward the west, so that the slopes on that side are gradual; the east fault plane has been carved into bold escarpments of exceeding grandeur. The elevations of the floors of the valleys between the successive mountain ranges indicate the relative power of the faulting forces. At the time of this great displacement of the earth's crust, titanic agencies were at work, and in regions of ancient volcanic activity deposits of precious ores are often found. This district is not an exception. In the early Eighties a gold stampede took place into the Saline Valley and the Ubehebe Mountains forming its eastern boundary. It is said that more than 2 000 people were camped in this valley during the height of the excitement. At that time it became generally known that there was a great salt deposit of remarkable purity in Saline Valley. A view of this deposit from the flanking mountains on the west is most impressive. The most important portion of the salt flat covers an area of approximately 1 500 acres, and the purity of the salt is enhanced by a natural refining process which is in constant operation, due to the remarkable fact that there is a series of large springs on the western rim of the salt flat, the water of which inundates the salt beds during the winter, and yields to the excessive evaporation superinduced by the summer heat. This retreat of the water deposits the pure salt crystals, which form when the brine reaches the point of saturation.

The salt claims located at that time have always been assumed to possess great value, provided transportation facilities could be obtained to ship the salt to the various markets.

Surveys had demonstrated the impracticability of building a railroad into the valley; the mountain barrier was so great that the cost of constructing such a line was excessive. Two methods of transportation were then considered. The first was by a pipe line over the Inyo Range, through which the brine formed on the salt flat could be

pumped to evaporating vats adjoining the railroad at Owens Valley. This project was examined very carefully, and estimates of cost were prepared, but it was found that although it was feasible for the transportation of brine, it did not afford a means of bringing in the supplies needed by the operations incidental to pumping. Accordingly, the merits of an aerial tramway were discussed, and, in order to ascertain the cost, and determine a suitable location for such a tramway, a survey was begun in April, 1911, under the direction of W. H. Leffingwell, M. Am. Soc. C. E. Several trial lines were run, and a location was selected in the latter part of May. Under the direction of Mr. C. H. Wickham, Field Engineer for the Trenton Iron Company, a final location was completed in July. The profile, together with the topographical surveys, were submitted to the Trenton Iron Company for consideration. An estimate was prepared, and the final contract was signed on August 14th, 1911, between The Trenton Iron Company, a subsidiary of the American Steel and Wire Company, and the Saline Valley Salt Company, a corporation of the State of Arizona.

This contract called for the preparation of the designs and finished drawings, as well as the wire cables, carriers, and machinery, for a Trenton-Bleichert tramway to carry salt weighing 60 lb. per cu. ft., the slope length of the line being 69 645 ft., the capacity 20 tons per hour, the elevation of the discharge terminal above the loading terminal, 2 450 ft., the carriers to have a volume of 12 cu. ft., and the speed of the traction rope to be 500 ft. per min. This resulted in a spacing of 525 ft., or 63 seconds, between contiguous carriers. More than 112 h. p. were required for its operation. The following items occur as prominent parts of the specifications:

| | |
|---|---|
| Patent locked coil steel track cable, $1\frac{1}{4}$-in...... | 13 850 ft. |
| " " " " " " $1\frac{1}{8}$-in...... | 55 450 " |
| " " " " " " $\frac{7}{8}$-in...... | 69 300 " |
| Special steel traction rope, $\frac{3}{4}$-in................ | 141 000 " |
| Special steel buckets, capacity 12 cu. ft., with protecting covers; and hangers, carriages, and patent compression grips.............. | 286 |
| Intermediate supports...................... | 120 |
| Track cable saddles........................ | 240 |
| Steel traction rope rollers, with shafts, bearings, bolts, washers, foot castings, etc........... | 240 |

The specifications also included the erection of the timber work and attaching the machinery; the equipment for one loading terminal, one discharge terminal, five intermediate control stations, twenty-one rail structures, twelve anchorage-tension structures, and one double-tension structure; tools for coupling and splicing track cables and traction rope; the necessary signal gongs for the control of the line when in operation; track cable oiling cars, traction rope oiling tanks, and water carriers.

The difficulties of construction on this line may be inferred from an inspection of the profile, Fig. 1. Aerial tramways have been built in the past so that the line traversed country possessing exceedingly rough topography, but each has been distinguished by the fact that it has been on the flank of a mountain range, instead of crossing it. It was desirable to find the shortest route between the salt fields and the railroad. The west side of the Inyo Mountains, though broken into a number of precipitous ridges, does not impose extreme difficulties to aerial tramway erection. The average elevation of the crest of these mountains is 10 000 ft. above sea level, and it runs in a northerly and southerly direction. However, there is a saddle in the crest line with an elevation of 8 500 ft., so that it was desirable to have the tramway pass through this gap. By utilizing the difference in elevation between the terminal stations, it was found that the 7 500 ft. required to pass the summit could be obtained by dividing the difference in altitude between the floor of Saline Valley and the summit into three 2 500-ft. sections; also, that the descent from the summit to Owens Lake could be made in two 2 500-ft. sections. The selection of a constant difference in the height of the several control stations was an important factor in deciding on a constant size of traction rope for all the sections. The use of intermediate control and driving stations added flexibility to the location of the line, as any reasonable horizontal angle can be made at such points.

The final location of the line is shown on the map, Fig. 2. It starts from the edge of the salt field, at an elevation of 1 060 ft. and runs south along the mesa for $2\frac{3}{4}$ miles to the foot of the eastern face of the mountains, which has an elevation of about 1 800 ft. The line ascends abruptly to Station 177 + 50, which has an elevation of 3 720 ft., where Control Station No. 1 is placed. The portion of the line from the loading terminal to Control Station No. 1 is designated

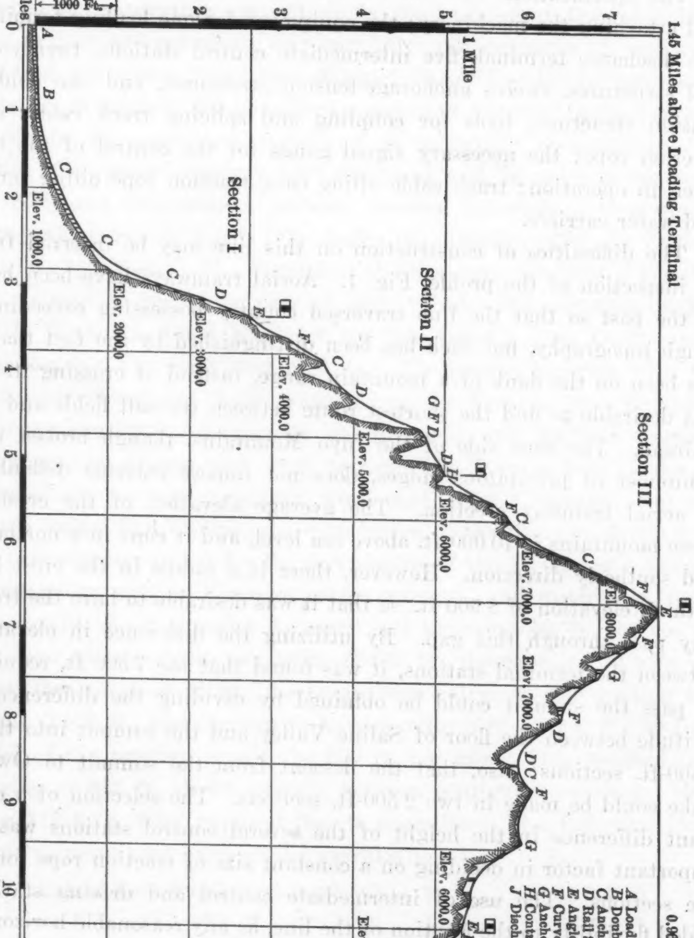



FIG. 1.—PROFILE AERIAL TRAMWAY, SALINE VALLEY SALT COMPANY, TRAMWAY, CAL.

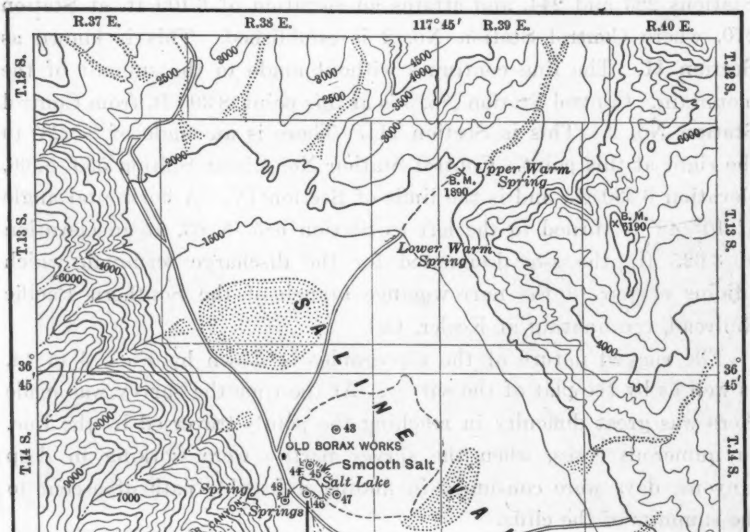



FIG. 2.—LOCATION OF AERIAL TRAMWAY OF SALINE VALLEY SALT COMPANY (U. S. G. S. TOPOGRAPHY).

Section I. A horizontal angle of 35° 30′ is turned to the right, or westward, at this control station. The line crosses Daisy Canyon between Stations 223 and 244, and attains an elevation of 6 100 ft. at Station 270, where Control Station No. 2 is established. This is known as Section II. The line continues without angle to the summit of the mountain. Control Station No. 3 is at this point, 8 300 ft. from Control Station No. 2. This is Section III. There is an angle of 2° 28′ to the right at this point. Control Station No. 4 is at Station 550 + 66, elevation 6 330 ft., and is the limit of Section IV. A horizontal angle of 10° 48′ is turned to the left to Station 685 + 63, at an elevation of 3 625 ft., the site designated for the discharge terminal, which adjoins a spur of the narrow-gauge branch of the Southern Pacific Railroad, terminating at Keeler, Cal.

The rugged nature of the topography is shown by Figs. 3 to 14, as well as by the plat of the survey. At the time the survey was made there was great difficulty in reaching the points traversed by the line. In numerous cases, when the survey parties were working in deep canyons, days were consumed in finding a feasible path of ascent to the summits of the cliffs.

Construction was started on September 1st, 1911, when the framing of the timber for the towers on Section IV was commenced. While the framing was in progress, new roads and trails were being built, so as to command the location of the line. At a point on the railroad about 3½ miles north of Keeler an old charcoal road traverses the west slope of Inyo Mountain to within about a mile of the summit crossing. The summit was connected to the old road by a new construction, and the old road was repaired and made serviceable. However, at an elevation of about 5 900 ft. there was a stretch of about 300 ft. of road having a grade of 25%, and the conditions were such that this grade could not be eliminated or improved at a reasonable cost. Thus 25% became the ruling grade for hauling the material needed for the construction between Control Stations 4 and 1. The maximum load which eight horses can haul on such a grade is about 5 000 lb. Heavier loads required ten or twelve horses. As the distance from Owens Valley to the summit was about 10 miles, a camp was established on the road about 3½ miles west of the summit. By this arrangement teams could make one trip per day from the railroad to the camp, leaving their loaded wagons for other teams, quartered at the camp,

FIG. 3.—SALINE VALLEY AND STATION 15. ANCHORAGE AND TENSION STATION 11.

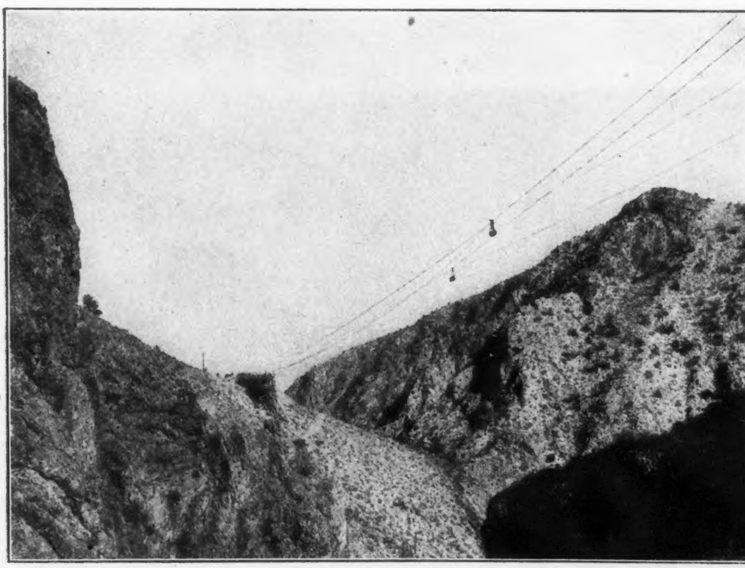


FIG. 4.—TRAMWAY CROSSING DAISY CANYON, CONTROL STATIONS 15 TO 16. LINE RIDER ON CARRIER.

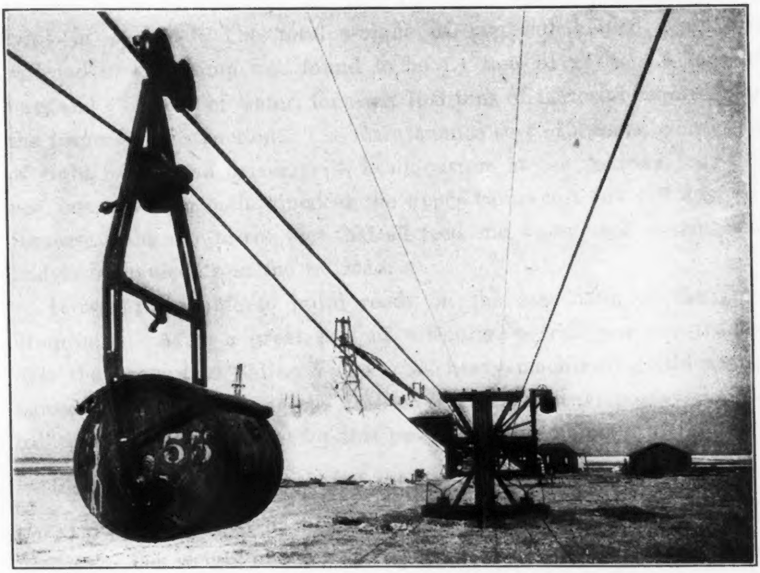


FIG. 5.—BUCKET LEAVING LOADING TERMINAL.

FIG. 6.—LOADED AND EMPTY CARRIERS PASSING ON 80% GRADE, SHOWING VERTICAL POSITION MAINTAINED BY BOTH.

to haul to the summit and unload. The camp teams could make three trips in 2 days. The total weight of material hauled from the railroad to this camp was found to be 1.1 tons of grain, 5.4 tons of hay, and 23.2 tons of water, for each 70.3 tons of material required for the tramway construction. The maintenance cost of a team, consisting of eight horses and driver, with headquarters at the railroad, was $15 per day. A team maintained at the upper camp cost $21 per day, the increase being due to the fact that all feed and water used at the camp had to be hauled from the railroad.

It was impossible to build roads on the east flank of the Inyo Mountains. After a great deal of difficulty, a trail was constructed over the summit to Saline Valley. As heavy machinery could not be moved by pack animals, it was necessary to construct temporary, double-cable, reversible tramways for this purpose. The diameter of the track cables used was $\frac{3}{4}$ in. and that of the traction rope, $\frac{3}{8}$ in. Suitable carriers were improvised by using two or more standard tramway timber carriers. The weight of the material transported by these jigbacks was approximately as follows: From Control Station No. 3 at the summit, for Structures Nos. 19, 18, 17, 16, and the towers in Section III, about 60 tons, at a cost of from $2 to $4 per ton, the difference being accounted for by the fact that the material was unloaded from the line as required: From Structures Nos. 20 to 15, a distance of 8 300 ft., 600 tons were moved, at a cost of $4.20 per ton; from Structures 15 to 11, 2 600 ft. distance, 375 tons at $2 per ton; Structures 11 to 9, 4 000 ft., 260 tons at $3.50 per ton; Structures 9 to 7, 2 700 ft., 210 tons at $2.90 per ton.

The cost per day for the crew required to operate, load, and unload these temporary cableways was from $35 to $42. The capacity of these lines, in tons per day, when operating, may be summarized as follows: From Structures 20 to 15, 13.3 tons; 15 to 11, 33.3 tons; 11 to 9, 23.8 tons; 9 to 7, 29.7 tons. If the capacity is stated in tons per day, including the time of putting up, taking down, and moving the jigbacks, the figures are as follows: Structures 20 to 15, 6.3 tons; 15 to 11, 19.3 tons; 11 to 9, 11.3 tons; 9 to 7, 14 tons.

From Chuckwalla Hollow, between Structures 4 and 5, a line was built to Control Station No. 1. It cost $35 a day for the crew, and took 6 days to build. One trip per day could be handled with this line,

at an average cost of $50. If the material was light, such as wood, water, etc., two trips could be made. This meant a labor cost of more than $25 per ton for handling motors and other heavy pieces of machinery from Structures 4 to 7. This expense is accounted for when it is recalled that the vertical lift is 1 900 ft. in a horizontal distance of 3 500 ft.

Saline Valley can be reached by a wagon road, approximately 55 miles long, which leaves the railroad at Big Pine. About 375 tons were hauled over this road at a cost of $35 per ton. All the cables required between Stations 1 and 2, together with the 75-h.p. motor, transformers, and heavy station material, were moved from the summit to a point in Daisy Canyon about 1 500 ft. west of Control Station No. 2. This material was transported on what is called a "go devil." This device consisted of a timber frame supported on an axle 8 ft. long which passed through two heavy wheels. The rear end of the frame rested on the ground, and was shod with iron in order to prevent excessive wear. On each side of the frame, near the rear end, steel hooks were arranged, so that two men, one on each side of the frame, could lift it free from the road when the gradient was light. On the other hand, these hooks were used to check the velocity of the device when traveling over ground that was soft enough for them to penetrate. The operators used both hooks in coming down steep places, and also assisted the team in steering the device. Fortunately, no road was required for the operation of this machine, as the canyon was fairly straight and the bottom free from ledge rock.

The material was moved to Control Station No. 2 on a tramway constructed with two carrying cables mounting four carriers. A different system was adopted for stringing the cables from the summit to Control Station No. 2, and also from No. 2 to No. 1. This consisted of coupling cables together and letting them down the mountain side by gravity. It was necessary to keep from three to five men at the leading end so as to put the cables on the traction rope guide sheaves of the towers and stations. The velocity of the cable was controlled with 4 by 8-in. blocks bolted together, so that the cable could be squeezed as it passed along. Each one of these extemporaneous brakes would resist a pull of about 1 000 lb. until a groove equal to the depth of the cable was worn in it. When this occurred the clamp was changed so

FIG. 7.—THE 2 400-FT. SPAN. STATION 7 IS AT THE TOP, UNDER THE CROSS.

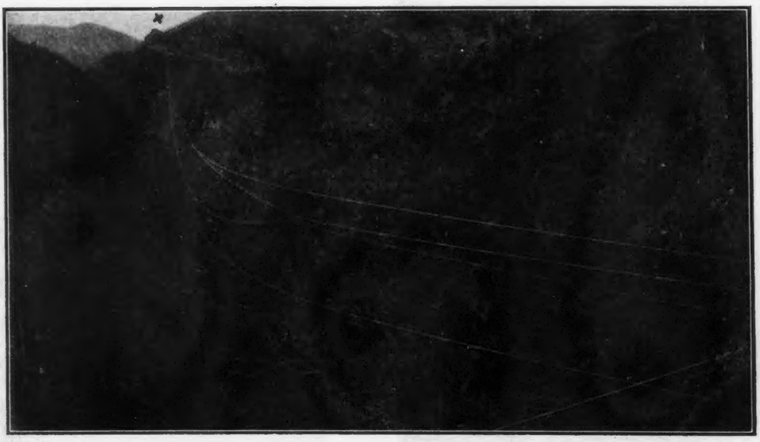


FIG. 8.—TRAMWAY CROSSING DAISY CANYON, STATIONS 10 TO 11; 2 260-FT. SPAN; 600 FEET ABOVE BOTTOM; 85% GRADE INTO STATION 11.

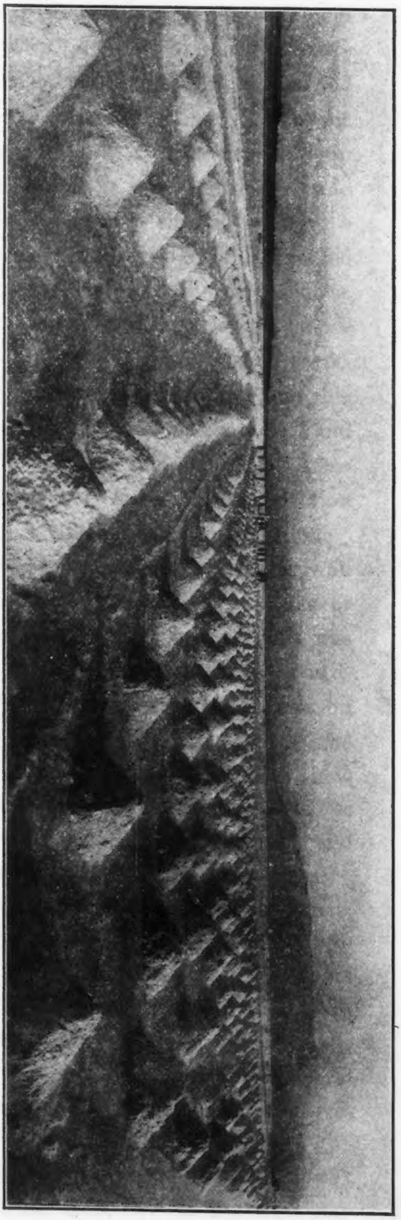


FIG. 9.—THE SALT FIELD.

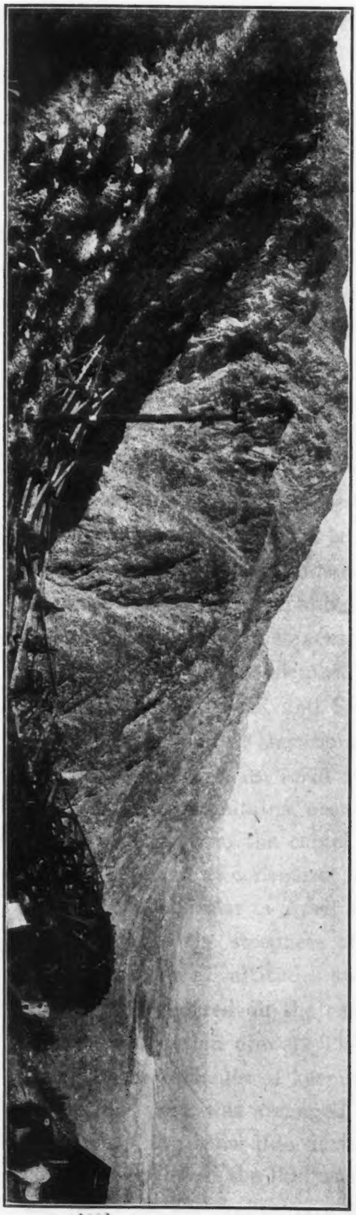


FIG. 10.—STATION 7, AND DISTANT VIEW OF SALT FIELD. NOTE THE REVERSE VERTICAL AND HORIZONTAL CURVES IN THE STRUCTURES.

that the cable would bear on a new spot. From Structure 4 to Control Station No. 1 the cables were pulled up with a winch driven by a gasoline engine. On Sections IV and V the cables were strung and placed under tension in the usual manner, except that on Section V the grade was sufficient to make it desirable to use the gravity method. In placing the traction rope for Sections III and II, it was threaded around the grip sheave at the summit station. The grip sheave acted as a driving sheave, due to the use of a brake on the reel, which imposed tension in the traction rope before it passed around the grip sheave. The traction rope was also passed through hauling clamps before reaching the grip sheave so as to hold it when splices were being made, or in cases of emergency. After getting the traction rope for one side to Control Station No. 1, the upper end at Station 20 was snubbed, and the rope for the opposite side was let down in the same manner. After the upper ends had been spliced, the ropes were cut at Control Station No. 2 and passed around the tension and grip sheaves of this station.

The towers and structures are carried on concrete foundations, and the water required for making the concrete was hauled in tanks or packed by mules for distances ranging from 1 to 7 miles. This was also true for the water required for the camps, except those at the railroad, in Daisy Canyon, and Saline Valley. All permanent camps on the east slope were at elevations lower than 6 000 ft., and those on the west slope lower than 7 500 ft., so as to be below the usual snow line. As the mountains are extremely barren, firewood had to be hauled or packed to the camps from the most convenient points, and these were often at considerable distances.

From October 1st to April snow storms are of frequent occurrence, and, owing to the steepness of the hillsides, constitute a menace to construction at all altitudes above 6 500 ft. In the spring occasional slides have occurred on the eastern slope, and have proved disastrous to the construction crews. The drifting snow was very objectionable, owing to the difficulty of keeping the trails open. Due to these several causes, the work was arranged so that it could be performed at elevations below the snow line during the winter.

The erection of the line was pushed energetically, and the tramway was ready for the preliminary test on June 10th, but the electrical con-

nections from the power-house of the Los Angeles Aqueduct, which is in Cottonwood Canyon on the opposite side of Owens Lake, were not completed until later. The first bucket of salt arrived at the discharge terminal on July 2d, 1913, and was the occasion for a great demonstration.

The placing of salt at the railroad was a great achievement, and taxed to the utmost the financial capacity of the Saline Valley Salt Company. The reasons for this stringency may be indicated as follows: After the contract was signed, the question of the location of the discharge terminal was an open one for the next 7 months. This indecision as to the terminus of the tramway affected the schedule which was prepared to guide the manufacturer in fabricating and shipping essential parts. It also deranged the plans of the construction crew. Nothing could be done in the making of surveys and the preparation of profiles, designs, or drawings until the site of the discharge terminal was known. Because of this confusion the completion of the contract was delayed beyond the intended date, so that the Salt Company was required to meet the expenses of a construction organization for a longer period than was contemplated in the original estimates.

The contract specified that the capabilities of the tramway should be demonstrated by an operating test of not more than 60 days, under the supervision of the engineers of the buyer and seller. To carry out this provision, active steps were taken by the field engineers to place the specified number of carriers on the line and to co-ordinate the various functions of the machinery, carriers, and signal systems so that the operating test could be undertaken as speedily as possible.

The line worked with great satisfaction when the buckets were loaded about two-thirds full. When efforts were made to carry full buckets, it was noted that the number of accidents due to runaway carriers and others slipping on the traction rope was excessive. An examination of the equipment was immediately started to ascertain the cause. After carefully canvassing all the agencies which might have influenced this unusual behavior of the carriers, the question of the weight of the salt as furnished to the tramway was raised, and an investigation was undertaken to determine the facts. The contract called for the transportation of salt weighing 60 lb. per cu. ft. The

FIG. 11.—TRAMWAY CROSSING DAISY CANYON. ALL PACK TRAINS MUST PASS THROUGH THIS NARROW DEFILE.

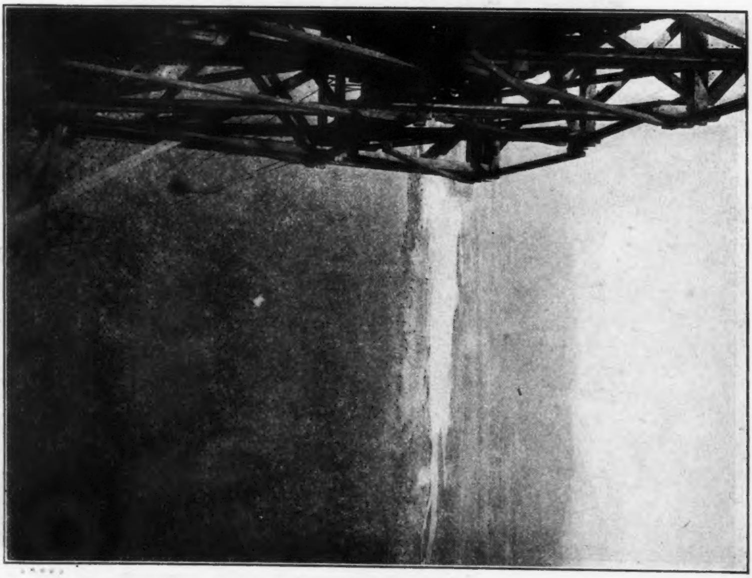


FIG. 12.—VIEW OF SALT FIELD FROM STATION 7. DIFFERENCE IN ELEVATION OF FIELD AND STATION = 2 600 FEET.

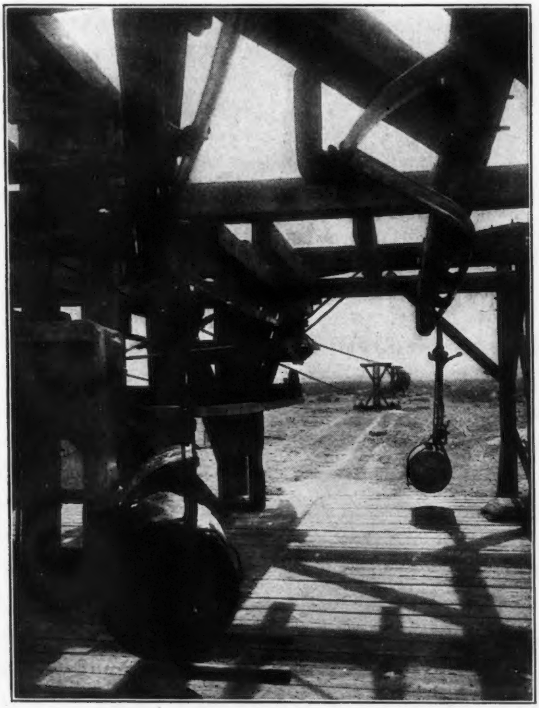


FIG. 13.—BUCKET OF SALT AT LOADING TERMINAL.

FIG. 14.—FLEXIBLE HOODS CONNECTING TRACK CABLE AND DOUBLE-HEADED TRAMWAY RAIL.

actual weights of the salt, as determined by careful tests at the salt field, were as follows:

| | | |
|---|---|---|
| Air-dried salt | 60 | lb. per cu. ft. |
| Damp salt | 65 | " " " |
| Salt sufficiently moist to agglomerate when squeezed in the hand | 68.5 | " " " |
| Salt when brine is dripping slowly | 83.3 | " " " |
| Salt when brine is dripping freely | 90.2 | " " " |
| When brine is running from salt | 94.2 | " " " |

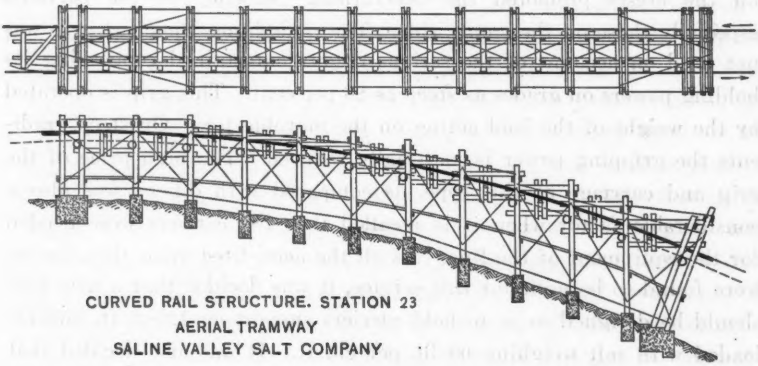
It was also noted that the average run of salt from the field was of the wet variety, so that its weight was in excess of 80 lb. per cu. ft. As the weight of saturated salt exceeds by more than 50% the figure stated in the contract, it is evident that the stresses imposed on the tramway had a similar range in value. Unfortunately, they were increments to the stresses contemplated in the original plan. The use of liberal factors of safety in design is again justified, for otherwise this line would have failed under the increased burden imposed. When these conditions were discovered careful observation of the behavior of the carriers showed that, when carrying dry salt they traversed the line from the loading to the discharge terminals without noticeable difficulty, but when loaded with wet salt, a few of the buckets would slip on the traction rope. The points where this difficulty was particularly pronounced were between Structures 5 and 6 of Section I; and between Structures 10 and 11 of Section II. There the track cables are as steep as, or steeper than, can be found on any operating aerial tramway using friction grips. In order to indicate the excessive gradients which confronted the designers of this tramway, Table 1 is presented. All gradients of less than 30% have been omitted. This table shows the station numbers of the approved survey, the numbers of the structures, the slope and length of the chord of the span, the number of loaded carriers supported on the span, and the slope, in angular measure and in percentage. These data will be found of great interest, especially to engineers concerned in aerial tramway design. Those who study Table 1 will realize the difficult problems these grades imposed on the designers of the tramway.

To know that this aerial structure is in successful operation is a source of deep gratification to its sponsors. The first great departure

### TABLE 1.—ELEMENTS OF THE AERIAL TRAMWAY OF THE SALINE VALLEY SALT COMPANY.

| Span. | | Slope of chord. | Length of chord, in feet. | No buckets in span. | Slope of cable. | Per-centage of slope. |
|---|---|---|---|---|---|---|
| From | To | | | | | |
| **SALINE TO SUMMIT.** | | | | | | |
| 135 + 50 (4) | 158 + 50 (5) | 20°06′ | 2 450 | 5 | 39°20′ | 81.95 |
| 158 + 90 (4) | 162 + 48 | 30°53′ | ...... | 1 | 36°17′ | 73.41 |
| 162 + 48 | 164 + 38 | 31°40′ | ...... | 1 | 36°28′ | 73.9 |
| 164 + 38 | 166 + 68 (6) | 32°16′ | Entering No. 6 | 1 | 39°45′ | 83.17 |
| 168 + 89 (6) | 173. | 30°27′ | 709 | 2 | 37°49′ | 77.61 |
| 173. | 176 + 03 (7) | 31°51′ | Entering No. 7 | 1 | 39°36′ | 82.73 |
| 178 + 57.3 (7) | 191 + 07.7 (8) | 20°18′ | 1 335 | 3 | 34°30′ | 68.73 |
| 191 + 34 (8) | 203 + 48 (9) | 18°19′ | 1 280 | 3 | 33°15′ | 65.5 |
| 219 T. | 223 + 08 (10) | 15°42′ | 500 | 1 | 20°37′ | 37.62 |
| 223 + 28 (10) | 244 + 10 (11) | 22°32′ | 2 260 | 5 | 40°27′ | 85.25 |
| 244 + 63 (11) | 247 + 22 (12) | 11°32′ | ...... | 1 | 19°08′ | 34.69 |
| 251 + 31 (13) | 265 + 91 (14) | 6°18′ | 1 459 | 2 | 21°24′ | 39.19 |
| 271 + 10 (15) | 284 + 50 T. | 15°44′ | 1 392 | 2 | 28°11′ | 53.58 |
| 284 + 50 T. | 289 + 11 (16) | 23°07′ | 500 | 1 | 31°00′ | 60.0 |
| 289 + 41 (16) | 295 + 31 (17) | 17°26′ | 513.5 | 1 | 25°23′ | 47.45 |
| 301 + 30 T, | 310 + 35 T. | 13°30′ | 931 | 2 | 22°36′ | 41.62 |
| 310 + 35 T. | 318 + 50 T. | 20°58′ | 873 | 2 | 29°36′ | 56.8 |
| 318 + 50 T. | 320 + 70 T. | 24°25′ | ...... | 1 | 29°20′ | 56.19 |
| 320 + 70 T. | 322 + 94 (18) | 24°32′ | ...... | 1 | 31°35′ | 61.5 |
| 323 + 33 (18) | 325 + 30 T | 20°06′ | ...... | 1 | 24°57′ | 46.52 |
| 325 + 30 T. | 330 + 00 T. | 21°18′ | 508 | 1 | 27°06′ | 51.17 |
| 330 + 00 T. | 336 + 00 T. | 23°54′ | 656 | 2 | 30°58′ | 60.0 |
| 336 + 00 T. | 338 + 29 (19) | 26°06′ | ...... | 1 | 33°30′ | 66.19 |
| 339 + 02 | 351 + 95 | 16°42′ | 1 352 | .. | 31°13′ | 60.6 |
| **SUMMIT TO DISCHARGE TERMINAL.** | | | | | | |
| 374 + 46 (21) | 377 + 20 T. | 18°31′ | ...... | 1 | 26°10′ | 49.1 |
| 377 + 20 T. | 379 + 00 T. | 18°33′ | ...... | 1 | 23°20′ | 43.1 |
| 379 T. | 390 + 00 T. | 12°31′ | 1 127 | 3 | 23°01′ | 42.5 |
| 396 + 15 (22) | 399 + 90 T | 19°11′ | ...... | 1 | 26°44′ | 50.3 |
| 399 + 90 T. | 400 + 90 T. | 17°14′ | ...... | 1 | 21°45′ | 39.9 |
| 400 + 90 T. | 413 + 50 T. | 10°46′ | 1 285 | 3 | 22°37′ | 41.6 |
| 421 + 51 (23) | 423 + 70 T. | 20°49′ | ...... | 1 | 25°44′ | 48.2 |
| 423 + 70 T. | 426 + 91 (24) | 19°48′ | ...... | 1 | 25°05′ | 46.8 |
| 427 + 13 (24) | 448 + 20 (25) | 8°10′ | 2 128 | 1 | 26°41′ | 50.3 |
| 497 + 40 (28) | 503 T. | 19°30′ | 594 | 2 | 28°23′ | 54.0 |
| 503 T. | 504 + 50 T. | 17°45′ | ...... | 1 | 22°26′ | 41.3 |
| 504 + 50 T. | 506 + 30 T. | 17°54′ | ...... | 1 | 22°41′ | 41.8 |
| 506 + 30 T. | 517 + 40 T. | 12°06′ | 1 135 | 3 | 22°45′ | 41.9 |
| 551 + 75 (29) | 553 + 10 (30) | 16°29′ | ...... | 1 | 23°14′ | 42.9 |
| 553 + 26 (30) | 556 + 50 T. | 19°27′ | ...... | 1 | 27°51′ | 52.8 |
| 556 + 50 T. | 557 + 50 T. | 19°08′ | ...... | 1 | 23°39′ | 43.8 |
| 557 + 50 T. | 558 + 50 T. | 19°57′ | ...... | 1 | 24°28′ | 45.5 |
| 558 + 50 T. | 561 + 34 (31) | 20°12′ | ...... | 1 | 25°21′ | 47.4 |
| 561 + 57 (31) | 580 + 68 (32) | 10°33′ | 1 945 | 4 | 29°26′ | 56.4 |
| 581 + 46 (32) | 584 T. | 15°35′ | ...... | 1 | 22°45′ | 41.9 |
| 584 T. | 588 + 10 T. | 14°18′ | ...... | 1 | 19°53′ | 36.2 |
| 588 + 10 T. | 590 T. | 13°19′ | ...... | 1 | 18°08′ | 32.75 |
| 590 T. | 591 + 28 T. | 13°21′ | ...... | 1 | 17°58′ | 32.4 |
| 591 + 28 T. | 592 + 50 T. | 13°35′ | ...... | 1 | 18°10′ | 32.8 |
| 592 + 50 T. | 593 + 77 T. | 14°02′ | ...... | 1 | 18°38′ | 33.7 |
| 593 + 77 T. | 595 T. | 14°37′ | ...... | 1 | 19°12′ | 34.8 |
| 595 T. | 597 + 50 T. | 14°50′ | ...... | 1 | 19°42′ | 35.8 |
| 597 + 50 T. | 600 T. | 14°37′ | ...... | 1 | 18°39′ | 33.8 |
| 600 T. | 605 + 80 | 11°44′ | ...... | 1 | 18°19′ | 33.1 |
| 631 + 38 (34) | 633 + 20 T. | 22°32′ | ...... | 1 | 28°17′ | 53.8 |
| 633 + 20 T. | 637 + 30 T. | 21°34′ | ...... | 1 | 27°09′ | 51.3 |
| 637 + 30 T. | 638 + 95 (35) | 20°53′ | ...... | 1 | 25°37′ | 47.95 |
| 639 + 30 (35) | 659 + 26 (36) | 9°27′ | ...... | 4 | 28°23′ | 54.0 |
| 663 + 97 (37) | 667 + 21 (38) | 13°11′ | ...... | 1 | 20°35′ | 37.55 |
| 667 + 76 (38) | 675 + 40 T. | 19°05′ | ...... | 2 | 27°58′ | 53.1 |
| 675 + 40 T. | 680 + 90 T. | 13°08′ | ...... | 2 | 19°30′ | 35.4 |

from standard tramway practice was made when the extreme difference in elevation between the loading terminal station and the summit was divided into three sections, and when two sections were used for the descent from the summit to the discharge station. Experience has shown that it is not economical to attempt to utilize traction ropes of large diameter. The total stress developed in such ropes when lifting 20 tons of material $1\frac{1}{2}$ miles vertically at the rate of 500 ft. per min. is so great as to prohibit the use of a single rope for this duty. Multiple traction ropes, arranged in parallel, are not to be considered when a more satisfactory solution of the problem can be developed. Accordingly, it was decided that the tramway should be arranged in sections so as to maintain a traction rope of moderate diameter, and to arrange



CURVED RAIL STRUCTURE. STATION 23
AERIAL TRAMWAY
SALINE VALLEY SALT COMPANY

FIG. 15.

the stations so that there would be a constant difference in their successive elevations. It is readily seen that this constant difference in elevation imposes practically the same stresses on the traction ropes of each section. The only feature that is not constant is the friction developed by the moving carriages on those sections which are not of equal length and consequently do not support the same number of carriers.

Unfortunately, experience has shown that, irrespective of the constancy of tension in two traction ropes in service, they do not maintain precisely the same diameter. Accordingly, friction grips which have been designed so as to maintain a uniform closure when adjusted to hold satisfactorily on the larger of the traction ropes will slip, all other conditions being constant, when attached to the second traction rope.

By this criterion it was necessary to abandon consideration of the well-known Webber grip for this service. The use of either the Bleichert overhead or underhung automatic grips was taken under advisement. The screw grip, such as Pohlig uses, was unsatisfactory because of the acceleration required to close the gripping lever, which moves in a plane parallel with the direction of motion; also because of the great difficulty in maintaining a proper limit for the throw of the closing lever when operating on ropes of different diameters so as to engage the detacher properly instead of under-running it, an accident which happens frequently on multiple-section lines where this grip is used. The overhead grip was eliminated from further consideration because the extreme tension of the traction rope in passing through structures on the crests promoted the overturning tendency of the carriers, especially those on the empty side. The underhung Bleichert grip was not used on account of the serious doubts entertained concerning its holding powers on grades as steep as 86 per cent. This grip is operated by the weight of the load acting on the movable jaw. On steep gradients the gripping power is materially reduced. The high price of the grip and carriage of this type, as compared with others, was also a considerable factor when it is recalled that 286 carriers were needed for the equipment of the line. As all the accredited grips then known were found to be unfit for this service, it was decided that a new grip should be designed so as to hold carriers supporting 12-cu. ft. buckets loaded with salt weighing 60 lb. per cu. ft. It was also decided that these grips should be of the trunnion type, so that the loads would hang vertically on all slopes; that the grips would compensate successfully without material loss of tractive effort for the different diameters of traction ropes found on the line; that the grips should be of the underhung, top-opening type; and that they should be of moderate cost.

When these grips were furnished and the line was operated, it was found that they fulfilled most of the aims of the design, but, owing to the great variation in the weight of the salt and the consequent increase in drag on the grip, some of them failed by slipping on the extreme slopes. In other words, the requirement for the constant adjustment of grips was too great a hazard to success to comply with the severe specifications imposed by the engineers of the American Steel and Wire Company, who ruled that, because of their uncertain performance when handling the heaviest loads possible, they must be replaced. This

conclusion was reached only after a considerable lapse of time from the date of arrival of the first bucket of salt at the discharge terminal, the reason being that the Salt Company experienced grave financial difficulties which, for about 2 years, prevented running the tramway in accordance with the contract.

Finally, the American Steel and Wire Company voluntarily took over the operation of the line at its own expense, so as to determine what defects, if any, existed, and what replacements should be made. As a result of this investigation, a new grip, called the Universal Wico, was perfected, which has demonstrated its fitness for the duty required. It is designed in strict accordance with the specifications previously outlined, and has sufficient gripping power on the traction rope to handle safely on the steepest grades a carrier loaded to its utmost capacity with the heaviest salt plus the weight of a line rider. It has the additional property of requiring a very light force on the closing lever, and is free from all jars so common to other forms of grips, due to the elasticity imparted to its action by the floating pivot of the closing lever. The grip requires about 40 lb. on this lever to close it. As the carrier has to be attached and detached five times during the passage from the loading to the discharge station, the troubles incident to these operations are five times as great as those on single-section lines, the usual form of the great majority of aerial tramways. In making a round trip on this line the carriers travel more than 26 miles, and a special lubricating system was attached to them so as to provide an adequate supply of oil for the wheels and pins.

Section I is exceptional. From the loading terminal to Structure 4, a distance of 13 550 ft., the line crosses a gradually rising mesa which serves as a fillet between the flat bottom of the valley and the abrupt escarpments of the fault cliffs of the mountain. From Structure 4 onward, the line rises 800 ft. with great abruptness. It is evident that in hauling carriers up a line of this contour the track cables and traction rope must be arranged so that the latter will never carry sufficient tension to cause it to rise above the track cables. Such a condition would be disastrous, as the carriers would be derailed. After careful computations, the tensions were determined so that this section operates perfectly.

Control Station No. 7 is unique in that it is equipped with driving mechanism, tension sheaves, guide sheaves, and a horizontal angle;

and the ends of the building are developed into a pronounced rail structure.

Control Station No. 15 is also equipped with driving mechanism, and presents a reverse vertical curve in profile.

The summit is unusual in that Sections III and IV are controlled from this point, both driving sheaves being mounted on the same shaft. (Fig. 16.) Therefore the power developed by the descending loads on Section IV assists in raising those on Section III.

As the carriers are being lifted on Sections I, II, and III, there is a possibility of the line reversing its direction of motion, due to the unbalanced weight of the carriers on the loaded and empty sides. Accordingly, each of these stations is equipped with a reverse preventer. This device consists of a large circular plate fitted with shoes containing ball races under the control of a centrifugal governor. When the governor is up to speed, the balls are restrained so that the plates run freely. When the line stops, the balls immediately nip the plate, and a reversal of motion is prevented.

Station 29 is also equipped with a reverse preventer to stop a retrograde motion of the line in case it is loaded with "back" freight destined for Saline Valley.

The drives of these stations are similar, except that at the summit two sheaves are mounted on the same shaft, as before mentioned. They consist of an 8-ft. grip sheave bolted to a brake run, 7 ft. 2 in. in diameter, with a 12-in. face, and this, in turn, carries an 8-ft. spur gear. The latter is driven by an 18-in. pinion mounted on a shaft fitted with a friction clutch belted to a 75-h.p., Allis-Chalmers, 3-phase, 2 200-volt, induction motor. On Section V this motor acts as a dynamic brake, instead of a driver. A grip sheave is one which carries a special rim, in which there are about ninety-six pairs of toggle jaws. These jaws are held open by gravity or by a special flat spring. When the traction rope passes around the sheave, the pressure developed on the bottom of the groove of these jaws is sufficient to cause them to close and grip the rope tightly. This action increases the friction between the sheave and the traction rope, thereby greatly promoting its efficiency.

The brakes are of the compound differential type, and are actuated by hand-wheels and screws; this method ensures that they will be applied gradually during an appreciable interval of time, so that the

line will not be wrecked because of sudden stoppage, as would be likely to occur with solenoid brakes.

The carriers are of special design, in that the hangers are made longer than is standard, so as to provide proper clearance between the traction rope guide rollers used in the rail structures on the ridges running transversely to the location of the tramway.

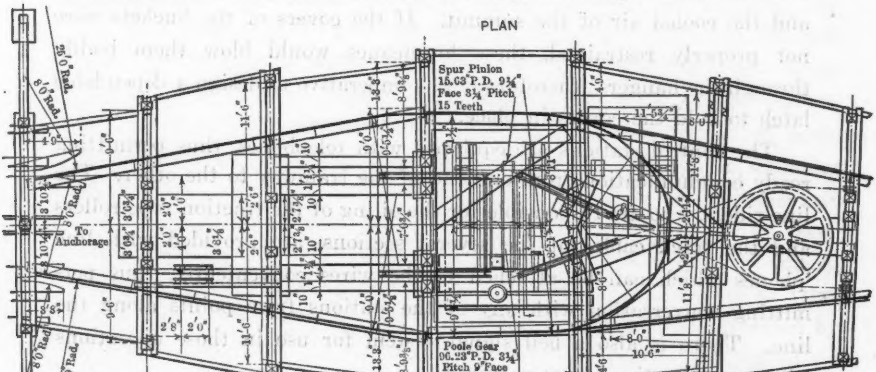



FIG. 16.—PART OF STATION 20, SHOWING DRIVES FOR SECTIONS III AND IV.

Owing to the extreme length of the line, it was necessary to cover the salt in order to protect it from contamination due to dust and grease which might drop from the track cable. To obviate the necessity of removing this cover whenever the bucket is dumped, the latter

is cylindrical, and revolves about its trunnions independently of the cover. Owing to the vibration of the carrier in passing through such a great number of stations and structures, it was necessary to provide a very positive keeper to hold the bucket latch in place. In the locality contiguous to the tramway site there are winds of great violence, promoted, no doubt, by the high temperature in the valley and the cooled air of the summit. If the covers of the buckets were not properly restrained, these hurricanes would blow them bodily through the hangers; therefore it was imperative to design a dependable latch to hold the covers in place.

The control stations are equipped with telephones, thus permitting ready communication from one end of the tramway to the other. The line riders, who are responsible for the oiling of the traction rope rollers and the track cables of the several sections, are provided with line 'phones which can be attached to the wires conveniently, thus permitting conversation with any of the stations from points along the line. There is also a bell signal system for use in those operations where conversation is not required.

The electrical transmission line is of the usual pole construction, carrying copper wires for a 3-phase circuit. The voltage is 33 000. The transformer stations are of the out-door type, and the voltage is stepped down from 33 000 to 2 200 for use in the motors of the driving stations.

The crew consists of two men each at the loading and discharge terminals, as well as at Stations 7, 15, 20, and 29. In addition to this crew, four line riders are required to keep the track cables and the traction rope rollers and guide sheaves in the structures and towers in perfect condition. There is also a foreman who exercises a general supervision of the line. The cost of labor and power for salt transported is 4.6 cents per ton-mile.

Several methods of gathering the salt have been tried with indifferent success. At present the plan is to flood the salt field by manipulating the water which is piped from the springs. This fresh water dissolves the salt rapidly. If the supply of fresh water is then stopped, the evaporation caused by the summer sun is excessive, and pure salt crystals gather in the brine. These crystals resemble flat plates, and are easily shoveled into conical piles about 2 ft. high and 3½ ft. in diameter at the base. Each pile is assumed to contain from 400 to

500 lb. The piles are arranged in rows approximately 12 ft. apart, the spacing between the piles being 6 ft. Mexican labor is found to be most satisfactory in gathering the salt, and each man is supposed to pile from 8 to 10 tons per day. Theoretically, the salt remains in piles until the brine has drained. As the mother liquor contains the majority of the impurities which might possibly contaminate the salt, this step is additional insurance of its purity. However, the salt does not drain freely, but remains damp for a considerable time. When the salt is gathered, a buggy of a special type is used. These vehicles are of galvanized iron and have a channel-like cross-section of body. Owing to the field being soft, the rear wheels are 20 in. in diameter and have a 12-in. face. The front wheels are 16 in. in diameter and have a 4½-in. face. The bed is 3 ft. wide, 8 ft. long and 6 in. deep. The axles are attached rigidly to the body, so that curves are turned by slipping the front wheels. These buggies carry from 500 to 1 000 lb. of salt, and are hauled across the field by a ⅜-in. cable operated by a gasoline engine. At the dumping point the wagons are pulled up an inclined plane until they assume an angle of approximately 44°, at which slope the wet salt slides freely into a hopper above a car which has a body 30 in. wide, 4 ft. 6 in. long, and approximately 30 in. high. The bottom of this car is hoppered both ways at an angle of 45°, so that the sides, instead of the ends, are used for doors. These cars are operated on a double-track railway, about ½ mile long, by an endless rope haulage system operating at a speed of 200 ft. per min. A 10-h. p. gasoline engine will move seven loads and seven empties on these tracks. The cars dump into a boot serving a drag conveyor which elevates the salt to a point sufficiently high to command the 50-ton bins at the loading terminal. Here the tramway buckets are filled by a lever chute of the ordinary type. The buckets are dispatched from the loading terminal at the tap of an automatic spacing gong. The rate established is fifty-six buckets per hour.

The tramway has carried material continuously at the rate of 23 or 24 tons per hour, and is an unqualified success. The stations, with the exception of the summit, are not housed so as to permit readily of winter operation, but the line is designed so as to prevent snow and other weather conditions from interfering with its operation.

The line is not designed for passenger service, but the line operators use it in preference to going on horseback over the rugged

mountain trails. A ride over this line is a never-to-be-forgotten experience. Starting from either terminal, the passenger rises rapidly to the summit, with a change from the torrid heat of the valley to the chill of the mountain heights. The descent is a reversal of this condition, so that it is difficult to dress properly to be comfortable throughout the trip.

Acknowledgment is tendered to the following gentlemen, to whose individual foresight, skill, and untiring energy are due the design, erection, and initial operation of the tramway: Mr. White Smith, President, Mr. Will Smith, Secretary, W. H. Leffingwell, M. Am. Soc. C. E., Chief Engineer, Mr. Daniel Kuhnle, Superintendent, Mr. M. O. Bolser, Electrical Engineer, and Mr. Harry Hilderman, Foreman of Construction, respectively, of the Saline Valley Salt Company; Mr. J. W. Smith, Local Manager, Mr. S. S. Webber, Chief Engineer (retired 1914), Mr. William Hewitt, Mechanical Engineer, Mr. S. W. Benson, Chief Draftsman, and Messrs. C. H. Wickham, L. T. Hays and T. J. Murphy, Field Engineers, respectively, of the Trenton Iron Company, and, later, Mr. R. H. Hall, Field Engineer, and Mr. George Hall, Foreman, for the American Steel and Wire Company.

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# PAPERS AND DISCUSSIONS



# MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS

BY J. F. PARTRIDGE, JUN. AM. SOC. C. E.

TO BE PRESENTED MAY 16TH, 1917.

SYNOPSIS.

**The object of this paper is to give engineers an idea of the difference between the various grades of wood pipes; to set forth a standard set of specifications for the assistance of engineers who have had no opportunity to become versed in their design; to safeguard those who contemplate building such pipe; and, further, to remove doubt from the minds of those who view wood pipe as one of the vagaries of engineering practice and a medium to be resorted to only in temporary and cheap work. If it can be shown that, to secure good results, the great difference in the quality of the materials used should be completely borne in mind, and if engineers can be led along a correct and standard course in the design and in the selection of these materials, this paper will have accomplished its object. To this end, specifications involving the latest and most approved practices are given in the Appendix.**

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The elements causing success or failure in wood stave pipe are taken up step by step, as follows:

1.—Kinds of wood used,
2.—Grade of lumber used,
3.—Method of curing lumber,
4.—Method of treating lumber,
5.—Location of pipe when built,
6.—Size and spacing of bands,
7.—Methods used in erection, and quality of workmanship.

The foregoing headings are discussed as applied to the two types now in use, namely, continuous-stave pipe and machine-banded pipe, and a plea is made for the adoption of uniform specifications, dividing each type into Classes *A*, *B*, and *C*.

The Appendix contains the specifications, for the two types and three classes of pipes, which are proposed as a basis for adoption by engineers.

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In 1898, the late Arthur L. Adams, M. Am. Soc. C. E., presented to this Society a paper* entitled, "Stave Pipe—Its Economic Design and the Economy of Its Use." This was the first important presentation of wood stave pipe design, and it brought forth a great deal of discussion. Mr. Adams prophesied the value that wood pipe would attain in hydraulic engineering, and was the first to give it its proper place with reference to cost, life, and capacity. He placed it first in economy of construction, first in carrying capacity, and second only to cast iron in length of life. These statements were rather startling to most engineers at that time, as they considered his deductions based on insufficient data. In 1906 Mr. Adams presented another paper† discussing the famous pipe of the Astoria City Water-Works. The discussion on that paper gave to engineers for the first time an idea of what could be expected of wood stave pipe. However, it was thought that a few more years must elapse before the real economy of such pipe could be determined. This was conservatism, for which engineers are noted, and proved to be a wise policy. Mr. Adams foresaw the value of wood pipe, but experience showed that all such pipe could

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 27.
† *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 65.

not be placed in one class. Development with years made it evident that careful design and selection of material were necessary, without which complete pipe failures would result. The indiscriminate use of various woods, which time showed to be unfit for good pipe construction, proved that it was wise to make haste slowly.

Though Mr. Adams foresaw the ultimate success to be reached by wood pipe, he did not foresee the rapid strides, and consequently the hasty, unscientific, and indiscriminate use of all kinds of materials; and these proved totally unsuited for pipe design, and threatened to turn success into failure.

In various engineering journals, in recent years, much space has been devoted to this subject, and the articles written have thrown considerable light on what had been but a hazy understanding in the minds of most engineers. The advantages and disadvantages of wood stave pipe have been discussed, but these discussions have only led to greater confusion and doubt as to the best way to use it. Consequently, a most valuable asset to hydraulic work has been shoved into the background and considered as a type to be used only in special cases.

In all the discussions it is noticeable that there are no references to cases where wood stave pipe has been constructed or operated successfully, or unsuccessfully, for a number of years. Actual cases where it has been in service long enough to give an idea of its durability are wanted by engineers. With this information, they can give some assurance that, if such pipe is used, it is the proper construction.

In most of the articles referred to, statements like the following are the general rule: "Experience shows that staves must be completely and continuously saturated, and that intermittent or partial saturation leads to decay;" "a stave pipe is extremely short-lived, even when made of the very best selected wood, under partial saturation, especially in warm, humid atmospheres." Warm, humid atmospheres are often encountered in localities in which engineers may have wood pipe under advisement, and conditions of partial saturation or drying out of the pipe during a portion of each year are often unavoidable. Such statements, therefore, are misleading, as the general impression is that favorable conditions for wood pipe are few, and, under all other than favorable conditions, it will probably be short-lived. This is not

true of well-designed wood pipe, and especially does it give a false idea of its usefulness when made from the proper materials.

A few cases that contradict the general impression regarding such pipe under conditions of intermittent flow and partial saturation are well illustrated by the following:

1.—Supply line of the Utah Lake, Land, Water, and Power Company, at Mt. Nebo, Utah. A half pipe or flume and a 48-in. pipe built in May, 1893. Intermittent and partial flow during a few months of the year. At maximum flow under 70 ft. head. Pipe entirely above ground, and for part of the way bracketed against a rock cliff, with exposure to the south and the full heat of the sun. Inspection in October, 1914, showed pipe to be without decay. Clear redwood staves.

2.—Discharge line from sugar factory of Los Alamitos Land Company, Los Alamitos, Cal. Mr. H. C. Lawrence, Chief Engineer. Built in 1902. Used only for 4 months of the year. Discharge for refuse from factory; operating under no pressure. Mr. Lawrence states that the line is in very good condition; a few bands show corrosion, and a good many have been replaced. He estimates that the pipe will have a life of 50 years. Clear redwood staves.

3.—Sewer for Palo Alto, Cal. Built in 1898. Continuous flow from one-half to three-quarters full. Pipe extends across salt marshes bordering San Francisco Bay; exposed at low and covered at high tide. Portions of line buried completely, half buried part of the way, and remainder exposed on the surface. Edwin Duryea, M. Am. Soc. C. E., in his discussion on Mr. Adams' paper of 1906, stated that this pipe showed not the slightest decay, though the bands had corroded badly. Air exposure, contact with the humus in the soil, saline soil, and partial saturation only, seem to have had no bad effect on the pipe, which is to-day in perfect condition and operating continually. A portion of the line buried in sandy soil had to be repaired a few years ago, some of the top staves requiring replacing, due probably to the sandy soil drawing out what little saturation these staves received. Clear redwood staves.

4.—Water supply pipe line for San Diego and Coronado, Cal. Built in July, 1900. 13 550 ft. of 40-in. and 26 300 ft. of 36-in. carry water from Otay Dam to these cities. Maximum pressure, 295 ft.; minimum, 150 ft. Pipe buried for entire distance in alkali flats, but

above ground where several deep ravines are crossed on trestles. Examination by writer in 1916 showed staves to be in perfect condition. Mr. O. D. Fees, Superintendent in charge of line during construction, accompanied the writer, and stated that the wood was considerably harder than when first put in. It is quite possible that solubles carried in the water have entered and have been deposited in the pores of the wood. This is redwood pipe.

5.—Leaching tanks in the plant of the Krieg Tannery, San Francisco. Built in 1859. A number of tanks above ground leaching into those below ground. Removed in January, 1914, found to be in perfect condition, replaced, and now in service. Redwood staves used.

6.—A redwood flume built in 1888 for the Cuyamaca Water Company, San Diego, Cal., has much of its original lumber in place to-day.

7.—Two 32-in. inverted siphons, one 20 and one 15 years old, in the line of the Yakima Valley Canal Company, North Yakima, Wash., which operates during the summer only, showed tapered ends of staves in upper part decayed when the pipes were torn out to be replaced by a 48-in. line to increase the capacity. Redwood staves were used in all three pipes.

The writer's criticisms of the articles relating to wood pipe will have to be modified because of the recent publication* of a paper by D. C. Henny, M. Am. Soc. C. E., Consulting Engineer for the United States Reclamation Service. In this paper Mr. Henny made the first attempt to segregate the various types and grades of wood pipe. He presents valuable data as to what can be expected of the average wood pipe, made from various materials, and operating under various conditions.

The discussion, however, should be carried still further, and the facts regarding manufacture and design that will give the pipe the expected life should be investigated and presented, so that engineers can determine intelligently the type that will best fulfill their conditions.

Wood pipe is too often classed as a whole, irrespective of the material from which it is made, no attention being given to the fact that there is as much difference between the various makes as between cast-iron and steel pipe, in fact, more. It is quite possible to make a

* In the *Reclamation Record.*

run of steel or iron with identical quantities of impurities, thus obtaining practically uniform products. Wood, on the other hand, is the most variable material known to the structural engineer, and is acknowledged as such. Yet in discussing wood pipe, no distinction is made as to quality, which depends on the kind of lumber used in the staves.

On work of any magnitude, where prominent engineers are consulted, conduits are generally chosen after deep study, and the results usually prove worth the expense of expert investigation. There are countless conduits, however, throughout the United States, where cheapness has superseded economy, and the resulting failures have shaken the faith in the type. Wood pipe has suffered the most. The many conduits with staves of inferior wood and poor manufacture have made engineers and others skeptical of this type of construction.

Wood pipe of the stave variety—and this is the only type considered to-day—was primarily a product of the West, although first invented and built in the New England States. The high freight rates on steel (which had to come from the East) made steel pipes very expensive, and the large quantities of timber available in the West made the use of wood an economical necessity. To build stave pipe, timber must first be available, and then the proper machinery to mill the staves, otherwise, it would not pay to use this type, except in those few cases where large installations warrant the cost of erecting machinery to mill the staves. Companies with timber holdings and mills of their own were naturally in the best position to manufacture such pipe. As a consequence, this business drifted into their hands, and they undertook it merely to sell lumber, only a few companies being formed to construct such conduits.

To-day it is possible for individuals to obtain materials and bid on wood stave pipe contracts, and such work is often undertaken at absurdly low figures in competition with experienced companies, which, knowing their business, are unable to secure the work except at a heavy loss. The successful bidder does the work to the best of his ability, but, as the building of continuous-stave pipe requires years of experience, he loses money, the pipe manufacturers are unable to keep in the business, and the purchaser secures a pipe that never proves a success. It is the duty of the engineer to protect his employer

against such conditions, and to do so he must be fortified with good specifications and must enforce compliance with them.

The elements causing success or failure in wood stave pipe include:

1.—Kinds of wood used,

2.—Grade of lumber used,

3.—Method of curing lumber,

4.—Method of treating lumber,

5.—Location of pipe when built,

6.—Size and spacing of bands,

7.—Methods used in erection, and quality of workmanship.

Redwood, fir, cypress, and pine are in general use, and make pipes of different characteristics. The pipe is also affected by the sap, pitch, or knots in the staves. The method of curing—kiln or air-drying—also influences the quality. Treating lumber with creosote, or surface painting, also affects the final result. The location determines to a certain extent the type of pipe to be chosen, and the size and spacing of the bands and the methods used in erection make a first-class or a useless pipe out of the materials available. If an engineer knows only the general methods of construction, and not the fine points, he cannot build a wood stave pipe line as well as a company which has had years of experience, and is likely to have trouble.

A discussion of the merits or demerits of such construction is misleading unless based on a clear specification. It is known, of course, that wood pipe kept constantly saturated will last indefinitely, but, as such cases are not always found, one must consider what will happen under other conditions. Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots. Under conditions of partial saturation, this lumber will not last, and, even with saturation, the pitch and sap will be the cause of deterioration. Most failures are attributable to this fact. There are conditions under which fir or pine will have a long life and give perfect satisfaction. For instance, erected on cradles, allowing the air to circulate freely around it, pipe will give satisfaction if the climate is dry, so that mosses, etc., caused by dampness, do not accumulate on the exterior. Pipe under heavy pressure in compact soil will last indefinitely. Mr. Henny has given the following tabulation:

| Wood. | Condition. | Years. |
|---|---|---|
| Fir....... | Uncoated, buried in tight soil..... | 20 |
| " ....... | " " " loose " ..... | 4–7 |
| " ....... | " in air.................. | 12–20 |
| Redwood.. | " buried in tight soil, loam, sand, and gravel...... | (More than 25) |
| Fir....... | Well-coated, buried in tight soil... | 25 |
| " ....... | " " " " loose " ... | 15–20 |

Cypress makes a most excellent and durable pipe, and is the only competitor of redwood with reference to length of life and endurance under alternately wet and dry conditions. If cypress is selected to eliminate sap, it probably is as long-lived as redwood; at least, it is near enough to avoid discussion. The disadvantages of cypress are:

First, the quantity of standing timber is extremely limited, and it is estimated by conservative lumbermen that all the commercially available cypress will be cut during the next 10 years. Thus, those who have cypress are constantly raising their prices to correspond with the advancing rise of stumpage.

The second disadvantage is in the wood itself. It grows in swamp land, and the butts of the trees are usually under water. A cypress tree is the product of four or five small trees growing together. The result is that the sap does not come as it does in redwood, entirely around the circumference of the tree, extending inward only 2 or 3 in., but it occurs throughout the clear part of the log, in streaks or strips. It is common to see clear cypress with yellow sap streaks running through the center at intervals of about 4 in. This has brought about the peculiar condition, that cypress has a grade higher than clear, and (the writer believes) is the only lumber which is thus graded. "Tank" grade is the highest in cypress, and contains knots but eliminates sap. Cypress knots are smaller and harder than those of redwood, and are not as detrimental.

It is extremely difficult to get cypress for pipes, because of the sap, and, where sap is eliminated, the price of the wood is so high that it cannot compete. Cypress pipes are rare.

Redwood is the best known material for wood pipe, and its longevity is excelled only by cast iron. The acid or other peculiar constituent of this wood acts as a preservative or micro-organism destroyer, and protects and preserves it.

The cases of redwood pipe already cited illustrate its adaptability, whether laid on the surface of the ground, partly or completely buried, or run through salt marshes or tropical swamps in direct contact with the soil humus. Direct exposure to the rays of the desert sun, and alternate wetting and drying, when the pipe is used intermittently in irrigation systems, do not lessen its efficiency.

A thorough study of the conditions under which a proposed pipe will operate and an investigation of the materials best suited to withstand these conditions, should be made before specifications are written. These important points should be kept in mind in order to insure specifications that will cover the conditions closely.

Engineers should first decide the nature of the conduit they intend to build; that is, whether it is to be a permanent structure or is to last only 5 or 10 years, after which time it is to be abandoned or replaced by a conduit of increased carrying capacity. Then the nature of the local conditions relative to the pipe line should be ascertained, including climate, humidity, temperature, extreme and average pressure, nature of soil, probability of the pipe being buried or laid on the surface, and other details; and then specifications for the materials can be written.

There is great necessity for uniformity in drafting specifications, and for an understanding of the requirements for securing pipe that will fulfill the needs of the proposed work. At present practically every piece of work is covered by specifications embodying different fundamentals. This is most noticeable in a comparison of specifications for various projects of the Reclamation Service. On some of these a distinction is made between redwood and fir pipe, bids being asked for coated fir and uncoated redwood; though in other projects no such distinction is made, these woods being placed on an equal basis. This question of the coating offers the largest field for disagreement, most engineers being of the opinion that both redwood and fir pipe should be painted in order to obtain good results; on the other hand, many who have had experience with uncoated redwood claim that painting it is unnecessary.

The thickness of the staves is another point of difference, and has been settled theoretically and practically with widely varying results. There is some difference of opinion as to the spacing of bands, depending on the assumed factor of safety, which factor in turn is determined by

the greater or less conservatism of the engineer. In the specifications for the staves is found the greatest divergence, and without justification. It is not evident why pitch and knots should be allowed in fir and not in redwood staves. Another objectionable feature in some specifications is the provision for rigid supervision of the bands, though adequate stress is not laid on the requirements for the shoes, which, after all, must be capable of developing the full strength of the band. The tongues are seemingly the smallest item of continuous-stave pipe construction, but, nevertheless, are by no means the least important. In machine-banded pipe there is absolutely no basis for the present-day so-called specifications.

The result is that some pipe lines are well, and some poorly, designed, the latter very often being the least economical. The purchaser, paying for what he believes to be the best type obtainable, secures a piece of work which proves a failure. These failures hurt the owner and undermine the faith in such construction. A great many engineers have little or no idea of how to design a wood stave pipe, and when it is necessary for them to draw up specifications they seek everywhere for information and acquire and compile a heterogeneous mass of data which are mostly useless.

For the assistance of engineers who have had no opportunity to become versed in wood stave pipe design, and to safeguard those who contemplate building such pipe, specifications should be standardized. The Appendix contains the specifications suggested as the foundation for a standard.

Sap and pitch in the staves mean a short life for the pipe, as deterioration will start first in sap wood, pitch seams, or pitch pockets, and spread rapidly to the clear wood. Pine and fir cannot be secured commercially without these defects, and, therefore, are fundamentally inferior to redwood, in which absolutely clear staves can be easily obtained. At repeated intervals, heavy applications of some protective paint with disinfectant qualities will allay the danger of deterioration in fir and pine, but proof that their ultimate life will equal that of redwood has not yet been obtained.

The thickness of staves should next be considered. Of course, the thicker the stave the better the pipe, but this has economical limits. A thickness of ⅛ or ¼ in., more or less, should not be the subject of controversy between engineers. The best criterion of the required thick-

ness of staves is actual experience. Throughout one section of pipe there will be staves of entirely different characteristics, including grain, resistance to percolation, and ease of penetration. Slash grain, vertical grain, quarter-sawed staves, heart wood, etc., all have their influence on the thickness required, and, with such a great difference in the characteristics of each stave, a small difference in thickness does not affect the quality of the finished pipe.

In designing staves there is more than the thickness to be considered. Economy is the other essential feature. In common practice, stock sizes of lumber are chosen which will give the most economical number of staves to the linear foot of pipe. For instance, for a 36-in. pipe, 2 by 6-in. lumber is chosen. A maximum thickness and width of stave is obtained from lumber of this size and a certain number of feet, board measure, is obtained in the cross-section of the pipe. If 2 by 4-in. stock is chosen, in comparison with 2 by 6-in., the result may be a saving in the board measure, but the cost of erection of the pipe will be increased considerably on account of the greater number of staves to be handled. If 2 by 8-in. is used, a saving in erection is obtained, but the greater waste in lumber offsets the saving in erection. The result is that 2 by 6-in. is the most economical size. The maximum that can be obtained from this stock piece is the thickness to be specified. It should be remembered, however, that this maximum will be less than that obtained by laying out the stave on paper, showing it cut from 2 by 6-in. stock of exact dimensions. The stock as it comes from the mills, dry and ready for stave manufacture, will probably measure not more than $1\frac{7}{8}$ by $5\frac{3}{4}$-in. Allowing enough for milling, the thickness of the stave will be reduced. Common practice and experience in stave milling should always be considered. If greater thicknesses are wanted, a higher price may be expected, as uneconomical sizes of lumber must be used, or a higher price must be charged to cover the selection of wider and thicker stock.

The stave has to resist the percolation and the penetration of the water. It should be sufficiently thick to prevent excessive percolation, and, at the same time, there should be perfect penetration. It is difficult to determine this thickness. If the staves have rings showing wide, alternate spaces of hard winter wood and soft summer wood, there will be great danger of excessive percolation, the water finding its way out through the soft wood between the hard rings. If the wood is very

hard, there will be great difficulty in the stave receiving complete saturation, due to the absence of capillary action. A soft wood will take up water like a blotter, but a close-grained wood will effectively resist percolation. This is very important in determining the lumber to be used.

Fir and pine, being hard woods compared with redwood, and being coarse-grained, having wide rings of hard and soft wood, enter the classification of woods giving excessive percolation, with slow and incomplete penetration. This is caused by the water passing rapidly through the soft summer wood, appearing in drops on the outer surface of the pipe, and of penetrating but slowly, and often through only a fraction of a stave, along the hard winter rings. The result is a stave showing percolation and incomplete penetration at alternate points throughout its cross-section.

Redwood is very soft and cellular, and pipe made from clear stock will be free from percolation and will receive complete saturation, even under very light pressure.

There is such a great variation in the quality, grain, and degree of hardness of even the same kinds of woods, that it is impossible to secure a pipe in which the penetration and percolation will be of the same degree in every stave. In the same section of pipe, soft staves with good penetration and no percolation will be found adjacent to hard staves with poor penetration and excessive percolation. It is obvious, therefore, that a refinement of stave specifications to the point of $\frac{1}{8}$ or $\frac{1}{4}$ in. more than a practical working thickness is entirely unnecessary.

In the Appendix practical working thicknesses for staves are given, and it is recommended that engineers give them their attention when drawing up specifications. By a practical working thickness is meant that which can be secured from the stock sizes of lumber making up the most economical pipe.

The best selection of a stave, therefore, is the result of experience with those thicknesses which give maximum penetration and minimum danger of percolation.

It should be remembered that fir or pine staves require greater thickness than redwood, in order to resist excessive percolation, and the result is not altogether beneficial, as the penetration is less likely to be complete.

The difference in required thicknesses of fir or pine and of redwood staves applies more particularly to machine-banded pipe, because the thicknesses for continuous-stave pipe are determined primarily by construction conditions, rather than by reason of penetration and percolation.

One objection to wood pipe is the danger that it may dry out if the water is drawn off. To avoid this, the staves should be thoroughly dry, so that, when properly erected and cinched tight, there will be no leakage. The pipe should be tight and stay tight. If wet staves are used, no swelling can be relied on for making a tight line, and the requisite pressure between the staves to prevent the passage of the water must be the result of cinching the bands. This is practically impossible. The lumber, therefore, should be perfectly dry before being used. It should be dried by the natural or air-drying process, not by the forced or kiln-drying process. By air-drying only, is perfect, sound, strong lumber obtained. Kiln-drying makes brittle and lifeless lumber. Air-drying requires time, and as lumber should be seasoned for at least a year for the best construction, a large stock of it should be available at all times.

The old method of drying lumber (in Maine and Michigan) was by the use of live-steam kilns. These are still used in the Northwest for drying fir and pine, as such treatment is necessary in curing pitchy and sappy woods. The kilns are large rooms, along the floors of which there are perforated steam pipes into which live steam is turned. The lumber placed in such a kiln is literally cooked. Redwood when first marketed had never been kiln-dried, but when the demand became too great for the supply of air-dried lumber that could be kept on hand, kiln-drying was adopted. Redwood treated by this process was flinty, and could be broken into splinters over the knee. Such methods of drying are now being used by some of the redwood mills, but lumber thus treated should never be allowed in pipe construction. The later method of kiln-drying is by indirect heating with steam. Steam is introduced into pipes laid on the floor of the kiln, and air with a certain humidity is admitted into the kiln at a given temperature, is heated by passing up around the steam pipes, and, rising through the lumber, removes the moisture. The air then passes down the compartments at the sides of the kiln where the water it contains is condensed, and the cooled air is again brought down to the heating

pipes. A circulation of air is thus effected by which the lumber is dried. This is far superior to the old method. The introduction of green lumber into a kiln and the forced removal of the water causes a forced and sudden hardening and closing of the pores, checking, splitting, and rendering the wood brittle. Air-drying is a natural seasoning, the slower the better, and is brought about by the wind blowing through properly stacked lumber. When securing lumber for pipe staves, there should be a strict investigation into the methods of drying used by the mills. For correct pipe design, only air-dried lumber should be specified.

In regard to the protection of the staves by applications of coatings of paint or disinfectant, little of value can be cited. Many claims are made for the benefits derived from various coatings, but sufficient data are yet lacking for reliable conclusions. It is certain that such protection increases the life of fir, pine, or other woods containing sap and pitch, but its merits on a redwood pipe have not yet been proved. Though uncoated fir and pine, except under conditions of complete and continued saturation, have proved short-lived, similar pipes coated with a mixture of tar and asphaltum have given far better service, and in many cases appear to be in perfect condition. More than this is not known. The oldest lines, on the other hand, made of redwood, have never been coated with any protective coating, and are still in perfect condition.

A coating, to be effective, should be applied diligently and often. At least two coats should be applied primarily, by conscientious workmen or by some pneumatic process. As in all painting, the personal equation of the workman is 75% of the job. A coating of at least $\frac{1}{16}$ in. should be the result of the first painting, and repeated examination should be made of the line, and the pipe painted every year or so.

Steel bands can be obtained to-day from practically all the large steel mills. These bands are manufactured according to standard specifications; if other specifications are used the cost is greatly increased, and very often it is practically impossible to obtain them. Engineers should bear this in mind, as they will save the pipe constructors much trouble and the purchaser much needless expense by using the standard specifications. The band commonly used is of mild, open-hearth steel, having a tensile strength of from 55 000 to 65 000 lb. per sq. in., with a button head at one end and at least 5 in. of cold-

rolled thread on upset ends at the other. The requirements for pipe bands are included in the standard specifications in the Appendix. Specifications often call for pure iron bands, but, as pure iron is made only in very limited quantities, mostly in Norway and Sweden, it is evident that it would be impossible to comply with such specifications.

The size of the band steel and the spacing of the bands on the pipe are, after all, the most important factors affecting the strength of the pipe. Common practice requires the bands to be spaced so that they will have a factor of safety of four against breaking due to tension caused by the water pressure, though some specifications call for a factor of safety of five. The latter requirement adds greatly to the cost without benefiting the pipe. The factor of safety of four gives ample protection against failure under water pressure through rupture of the bands, but the point to be borne in mind is that the pipe may fail on account of the bands sinking into the wood and allowing the longitudinal joints between the staves to open, thus causing leaks. The failure of the pipe in bearing, however, in pipe more than 10 in. in diameter, is prevented if the bands are spaced with a factor of safety of four in the tension formula. The two formulas to be used in spacing the bands on the pipe relate to the tension in the bands and the bearing of the bands and staves. In the bearing formula the value to be determined by experiment is the strength of the wood in bearing. This has generally been taken to be greater for fir than for redwood, but experiment shows that staves in a saturated condition, as would occur in a pipe in place, have practically the same strength in bearing. This approximates a working stress of 800 lb. per sq. in. If the spacing of the bands is checked by both formulas it will be found that all pipes more than 10 in. in diameter will be designed according to the tension formula, and that the bearing will be amply cared for under this condition.

In the design of every pipe line there will be two or three sizes of bands that may be used, with their corresponding spacing, and it is necessary to choose the most economical. The smaller the band the closer the spacing, and the closer the spacing the better the pipe. Economy limits this to a certain extent, as the smaller bands cost more than the larger ones, and erection costs increase with the number of bands handled. A certain maximum spacing should not be exceeded, in good pipe design, and the size of the band may be cut down to hold

this spacing to a minimum, maintain the factor of safety, and still give economical erection. To hold to the maximum spacing with a large band wastes metal, but may be found to be most economical on account of the higher price of small bands. Erection has an important influence on the size of the bands on light-pressure pipe with the maximum allowable spacing, because, if small bands are used, it may be found impossible to cinch them tight enough to prevent seam leaks, and the threads on a great many bands will be stripped before the staves can be drawn together tightly enough to prevent such leakage. Heavy bands are required to draw the staves together, but after they are in place the initial tension in the bands is dissipated, and the only stress is that due to water pressure.

By no means the least important feature is the design of the shoes for cinching the bands tightly in place. These should be stronger in body than the bands, in order to develop the full strength of the latter. There should be a thorough investigation of the shoes, and, unless a special test is made, those that are standard and have been tested repeatedly should be used. It is folly to have bands spaced with a high factor of safety and then use shoes which are weaker in design than the bands they hold together. A case is known where this happened, and approximately 390 tons of steel were absolutely wasted. On some recent municipal work for a large western city a similar case of inferior shoes has lessened the efficiency of the conduit.

The tongues prevent the staves from working out at the butt joints, and are intended to form an effective water seal where the staves butt together. They are generally made of band iron, 1½ in. wide, No. 12 or No. 10 gauge, or ⅛ in. in thickness, cut ⅛ in. longer than the width of the stave measured along the slot. This allows $\frac{1}{16}$ in. to project into each adjoining stave, effectually preventing water from working around the joint. When the pipe is first cinched up, prior to rounding out the staves to the true circle, these tongues are set farther than this $\frac{1}{16}$ in. into the adjoining staves. Then, when it is attempted to round out the pipe, that is, hammer the stave from the inside into proper position, the tongues will tear the staves badly, and decay first starts where lumber is bruised or torn. Contact is all that is needed for a perfect water seal, and if the tongues are cut $\frac{1}{16}$ in. longer than the width, that is, cut to allow an initial projection of $\frac{1}{32}$ in. in the adjoining staves, a water-tight joint will be secured with less damage

to the wood during the rounding out of the pipe. There is little likelihood that the tongues will rust out, as air does not reach them readily; however, they are generally coated with an asphaltic base paint.

The rod of the least diameter that can be used on light-pressure pipe, with maximum spacing, can only be determined by experience, and should not exceed the sizes given in Table 1.

TABLE 1.

| Diameter of pipe, in inches. | Minimum size of rod, in inches. | Maximum allowable spacing, in inches. |
|---|---|---|
| 12 to 24 | 3/8 | 10 |
| 24 to 36 | 7/16 | 10 |
| 36 to 48 | 1/2 | 10 |
| 48 to 72 | 5/8 | 10 |
| 72 | 5/8 | 8 |
| 72 to 96 | 3/4 | 10 |
| 96 to 132 | 3/4 | 8 |
| 132 to 144 | 3/4 | 6 |

When the maximum spacing is used, extra bands should be placed over the butt joints to reinforce the pipe at these points. Many pipe failures have resulted from allowing too wide a spacing as a maximum, and such conduits, under a heavy back-filling, failed by the arch of the pipe collapsing. A maximum spacing of 18 in. has been used, but this does not make a pipe. A spacing of 12 in. has been used with success, but the best results are obtained with a maximum of 10 in.

The coating for the bands and shoes is determined by the conditions and the life to be expected of the pipe. For the bands of fir and pine pipe an asphaltum coating is sufficient, as the bands will outlast the staves. On redwood, contrary to the general impression, the life of the pipe will invariably be the life of the bands. The life of the pipe, therefore—or its economy—depends on the coating first applied to the bands; in after years the pipe should be well inspected and the bands repainted and replaced when necessary. A coating having an asphaltic base is most commonly used, and gives perfect satisfaction. Under normal conditions bands thus coated last from 10 to 15 years, after which time it will probably be necessary to replace them occasionally, although, as a whole, they will outlast steel pipe, as the metal is concentrated in a round band which presents a minimum surface to corrosion. Covering with red lead is recommended for severe conditions, bands having been found as bright under the red lead as when first erected, even after 26 years' service. In Central America coating

with red lead is found to be well suited for protection against the ravages of tropical climatic and soil conditions, and exposure to the salt air during steamer transit. Galvanizing may be used, but, on account of its excessive cost, is not common. In the Hawaiian Islands galvanizing is used exclusively, together with redwood lumber, and such design has been found to give practically the only pipe that will stand up under the conditions there.

No attempt will be made to outline the best methods of constructing wood stave pipe, because there are so many details that require attention and experience that, unless an engineer is familiar with such work, he will do better by securing the services of reliable pipe constructors.

Machine-banded or wire-wound pipe has come into use since Mr. Adams presented his last paper on wood stave pipe, and has found a ready market in the West, on account of its economy. It is factory made, in sizes from 2 to 24 in. inside diameter, and is designed for heads up to 400 ft. The sections are from 8 to 24 ft. long, and have the necessary couplings or collars for connecting them.

Since this type made its appearance, some time ago, there has been practically no improvement, and the old specifications and methods of manufacture are still followed. In spite of its many imperfections in design and manufacture, this type has made a wide field for itself, and is found in every branch of hydraulic work. Unless steps are taken to correct its weaknesses, however, it will rapidly lose favor on account of its numerous failures. These failures have not been altogether the result of poor manufacture, but have been due to an endeavor to reduce the cost. This was done to such an extent that good manufacture and design were impossible.

It is subject to the same criticism as continuous-stave pipe. Conduits of poor design and poor material, or material unsuited for the work, have given the impression that wood pipe as a whole is unsatisfactory and short-lived.

Modern machine-banded pipe is made with heavy staves, generally kiln-dried, banded with galvanized wire, from No. 6 to No. 00 gauge, spaced according to the pressure. The wire is securely fastened to the pipe with pressed-steel clips or staples, or both. Each section is dipped in asphalt and rolled in saw-dust, the asphalt effectually covering the wire and staves, and the adhering saw-dust permitting it to be handled

readily. The sections of light-pressure pipes are joined with inserted or slip-joint connections, being sometimes reinforced with a steel band equipped with a shoe for cinching tight. On high-pressure pipes (generally for more than 100 ft. static head) collars are used. These collars are made in a manner similar to the pipe, the sections being tapered and driven firmly into them. On pipes of large diameter, operating under heavy pressure, the collars have individual bands fitted with shoes for cinching. Riveted steel or cast-iron collars, with or without bells, are also used.

The foregoing describes the pipe generally made by all manufacturers of this type, and represents standard practice. Numerous failures have rendered the recommendation of this type doubtful. If investigations were made into the actual causes of the failures, the reasons would be plain, similar designs would be avoided, and there would be rigid inspection of the manufacture. In outlining the results of the methods of manufacture, it will be well to mention the reason that fir pipe is the butt of the criticism. Fir is the pipe that has failed, the oldest lines having been built not more than 10 years, the greater number being of comparatively recent date. When these pipes were made, fir was chosen as it was the cheaper wood, and there was no criterion as to longevity. The greater number of failures possibly originated at the joints. The outer edges of the staves in the collars, when wood collars were used, decayed rapidly owing to the fact that fir needs saturation for preservation, and saturation was not secured at those places. Cast-iron and steel collars were the remedy, but have not proved successful, owing to their high cost, the increased weight, and the difficulty of making tight connections and plugging leaks. Riveted steel collars can be used to advantage on fir or pine pipe, as they will last as long as the staves, in which the sap wood decays rapidly.

Redwood collars of the individual banded type are used for repairing pipes which have failed at the joints. The decayed fir collar is cut off, and the redwood staves are put in position and cinched tight. This method requires no further attention, and can be applied successfully at any joint where there is decay. The wire is cut away and securely stapled, and the redwood collar is put in position.

The use of the inserted or slip-joint pipe is not to be recommended. Such a connection weakens the end of every section, because nearly one-half of the shell of the pipe is cut away to make the joint. A

reinforcing rod is often used to draw the joint tight, but if the male and female tenons are eccentric, leakage cannot be avoided. There is also great danger of injury to the pipe by handling, before and during shipment, as well as in laying; the weakened ends are not reinforced, and often split off with rough handling. The collar connection is to be preferred, as it will insure a better and stronger pipe, and the greater length of life will warrant the increased cost.

Most of the serious failures have occurred when the water has been drawn out of the conduit for any length of time; this has caused the staves to dry out and the pipe to fall to pieces. Failures by the galvanized wire breaking, mainly at the twist splices, are serious, as the entire section of pipe on which such a splice occurs must be removed.

As a remedy for failure by decay, machine-banded pipes are now painted or dipped in a protective coating, but the same conditions exist here as with the continuous-stave pipe, and pitch seams and sap wood will cause failure, even in coated staves. Coating not only adds to the cost of manufacture, but increases the weight materially. Machine-banded pipe, being essentially a factory product, its cost is affected greatly by freight rates, as shipment from factory to site of erection determines the economy of its use in a great many cases. In this type redwood has a distinct advantage over fir or pine, as it is unnecessary to apply an artificial coating to preserve the staves; therefore, having no coating of tar or asphalt, and well-seasoned redwood being very light, it has a very low shipping weight. The coating often serves to cover defects in material and manufacture. Fir and pine pipe should be inspected rigidly before acceptance; and redwood pipe should be left open to inspection and thus save the difference in weight and the cost of dipping. Painting the wire is useless. When the pipe is built it is impossible to be so careful in handling that none of the paint will be scraped off. The wire exposed in one spot leaves a weak place at which corrosion may start. As the wire is wound under heavy tension, there can be no protective coating on it where it touches and is embedded in the wood; for that reason only a small part of its surface is coated.

To prevent machine-banded pipe from drying out and collapsing, thorough drying of the stave material and proper winding are necessary. By the use of thoroughly dried wood, wound under heavy tension in the wire, with close spacing to draw the staves together, securely and completely, high-grade pipe is obtained. Tests in winding redwood

pipe show that a tension of 25 000 lb. per sq. in. in the wire embeds it securely in the wood and draws thoroughly dry staves together properly without crushing the fiber under the wire or along the edges of the staves. A tongue slightly longer than the groove also assists in making the pipe water-tight when exposed to the sun after the water is drawn off. Exhaustive tests at the plant of the Redwood Manufacturers Company, at Pittsburg, Cal., showed that this tension produces a pipe having greater strength, in resisting possible over-loads, than if wound under lesser tension, and that higher tension crushes the wood fibers. This initial tension in the wire entirely disppears after winding, and the ultimate tension is that due solely to the water pressure.

The secret of correct manufacture is the thorough seasoning of the wood. Such a pipe can be laid directly on the surface of the ground and exposed to the heat of the sun without injury. A slight tongue and groove in the sides of the staves prevents their displacement if they shrink. This should occur only to a slight extent under most severe conditions, if the staves are properly dried and a proper process of winding is used.

Kiln-dried wood should not be used for machine-banded pipe. Pitch and sap should not be allowed, nor should untreated or uncoated fir or pine be used. Redwood does not need treatment to insure a life at least as long as treated fir or pine. Fir or pine pipe should be supplied with cast-iron collars with bell hubs for caulking. On redwood pipe redwood collars may be used, and as the wood does not rely on saturation for preservation, machine-banded and continuous-stave collars may be used to advantage. Inserted joint connections may be used, but will not give as good service as collars.

In choosing a thickness of stave, it is only necessary to use that which will resist percolation successfully and can be built into a pipe. Staves must have sufficient thickness to resist the stress caused by the high tension in the wire during the process of winding. If they are of redwood, there will be no danger of decay within 15 to 25 years, which is the life of galvanized wire.

For more than 18 months the writer has investigated tests of machine-banded pipe made at the plant of the Redwood Manufacturers Company, and the results have been most startling as well as gratifying. Continued experiments on various sizes of pipes under various pressures have shown that redwood pipe made according to the specifications for

Class *A*, of the given thickness of stave, and wound with the stated sizes of wire, will be absolutely water-tight, and, if designed with a factor of safety of four against the wire breaking and a value of 800 lb. per sq. in. for bearing, it will withstand successfully a 200% over-load of the pressure for which it was designed. An 8-in. pipe, with a shell only $\frac{11}{16}$ in. thick, and wound with No. 8 wire (0.162 in. in diameter), with a spacing of $2\frac{3}{8}$ in. from center to center, to withstand a 75-ft. head, has operated successfully under a greater head than 200 ft. Further, this same pipe, wound for various pressure heads, has been connected to the boiler feed pump, in the engine-room at the factory of the Redwood Manufacturers Company, and subjected to pressures varying constantly between 15 and 75 lb., has withstood, successfully and is still operating with over-loads as high as 80 lb., and has shown no leakage.

Wood pipe failures, when occurring in the body of the pipe, generally appear first along the longitudinal seams, which open up, allowing leakage. This is caused by the wire sinking into the wood; in other words, the bearing between the wire and the wood being destroyed, the staves move outward and the seams open. When this occurs the pipe is a failure. After such failure the staves return to their normal positions on release of the pressure, and the pipe will still operate successfully under the pressure for which it was designed.

It makes no difference whether the thickness of a stave is 1 or 2 in.; after the outer $\frac{1}{16}$ or $\frac{1}{8}$ in. has decayed, the staves move outward and the pipe fails. With thin staves, however, which can receive more perfect saturation, the maximum life is obtained.

The process of winding the wire on the pipe and drawing the wood together with the proper tension in the wire, actually determines the thickness of the staves. If the latter are too thin, they cannot be drawn into a firm seat against each other, but will buckle; the limiting thickness must be determined by experiment. During winding, a constant and uniform tension should be kept on the wire, drawing in the staves sufficiently to make all joints absolutely tight without crushing the wood. The closer the wire is spaced in this winding the better the staves are drawn together, and the tension required to do so is a minimum. A gauge, registering the actual tension in the wire, should be directly in front of the operator of the winding machine. The

tension varies, of course, with the diameter of the pipe and the spacing of the wire.

It is quite proper to give some guaranty of wood pipe design to the purchaser. When buying cast-iron or steel pipe, the head the pipe will withstand is known, but in wood pipe design and manufacture there are so many uncertainties that, without some guaranty of its strength, an engineer is at a loss to know its quality. He can check up the size of the wire and the spacing, but knows nothing as to the care in manufacture. If a pipe, guaranteed, say, for 50 or 100% over-load, can be obtained, the engineer then has a basis on which to work, and this is the ultimate method of correct manufacture to be expected. To secure a theoretical and practical basis for the design of machine-banded pipe and to determine an over-load for a guaranty, was the object of the tests just mentioned.

A radical departure from customary methods of pipe design is necessary to secure desired results. It must be remembered that, in mending a hoe handle, the farmer takes a small fine wire and wraps it as close as possible around the fractured part. He does not take a heavy wire and wind the handle with wide spacing. The correct principle of pipe winding is similar: Use small wire closely spaced, giving sufficient steel with such spacing as to insure a factor of safety of at least four against rupture under tension in the wire. A closer spacing is required on small pipe to prevent the wire from sinking into the wood because of insufficient bearing area. The use of small wire increases the bearing area between wire and wood. The reason for this can be readily understood, but the lasting quality of the small wire has to be considered.

Repeated tests and consultations with high authority on wire manufacture, and a study of the reasons, would show that the smaller wires are as well protected with galvanizing as the larger ones, and, if anything, a little better. The Western Union test, taken as a method of comparing the galvanizing on various sizes of wire, gave results which favored the smaller wires. The authorities, who are the wire manufacturers, state that the smaller wires are better galvanized because more care is taken with them in order to secure a product of the very highest grade. The reason is that, on account of the smaller wires being in greater demand, more care is taken to secure a uniform and high-grade product. In any factory output, market conditions

must be considered, and in the wire market small wire is of the best quality. Another reason for the use of small wire is the danger of destroying the galvanizing on large wire when winding pipe of small diameter. It is common to find a path of spelter directly under the pipe winding machine when winding small pipe with heavy wire, the result of the galvanizing spalling off. It is also found to spall off in splicing with the old twist, or Western Union splice. Splicing is necessary when coming to the end of the coil of wire while winding a section of pipe, and is of frequent occurrence when winding heavy-pressure pipe where close spacing is required. In making this splice the wire is twisted around its own diameter, and this injures the zinc coating.

Maximum efficiency is secured where there are no splices. Electro-welding with re-galvanizing has been tried, but the re-galvanizing is unsatisfactory, and, until better methods are obtained, splices should be eliminated. All fastenings holding wire in place on pipe should be galvanized. Pressed-steel clips will rust if not protected.

The essential feature for success in machine-banded pipe is the proper use of the proper materials. The wrong use of a good material will be as productive of failure as the use of poor material.

With high-grade lumber and a high-grade wire, with which every precaution is taken for protection against corrosion, and with scientific methods of pipe winding, thinner staves and smaller wire may be used. If reliance can be placed on the manufacturers, such methods will result in economy.

The foregoing comments and suggestions are not intended to serve as a theoretical basis for pipe manufacture, but to point out the methods in use to-day by the various manufacturers. The detrimental features are pointed out, the scientifically based principles are outlined, and the specifications in the Appendix are suggested for securing uniform practice, thus enabling engineers to know what type they will obtain when they call for bids.

The specifications should be based on the use to which the pipe will be put. No engineer would think of calling for 1:2:4 concrete for rough foundation work, where water-tightness and strength are only secondary considerations. Neither would he specify a 1:3:6 mix for concrete conduits, where both density and maximum strength are required. It is the same with wood pipe. For cheap lines of short

duration, such as construction work, when the pipe will be abandoned after a few months or a year, a low-grade uncoated fir or pine pipe may be chosen. On temporary work requiring a life of 5 or 6 years, a good grade of uncoated fir and pine pipe may well serve. For 8, 12, or 15 years' service, a good, properly painted fir pipe may be used to advantage. For permanent work, redwood should be selected, or fir or pine of high-grade staves kept saturated and well painted.

Owing to the various grades of work in which fir or pine pipes are used, alternate specifications are given, but, as redwood would only be selected for permanent work, and for some conditions in which fir or pine would fail, such as in low-pressure work, only one specification is given for this wood, and this covers the highest grade.

It is sincerely hoped that this paper may lead to a definite idea of the merits of wood pipe and the adoption of uniform specifications.

In writing specifications for wood pipe, various pipes will be designated by classes, based on the nature of the work they are to do.

### Continuous-Stave Pipe.

*Class A.*—A pipe having a maximum life, under all conditions, and this will be 25 years when receiving no care whatsoever; a life greater than 25 years, if under continuous operation; and a probable life of 50 years, or more, if in continuous operation under at least a moderate head, if the bands are given attention and corroded ones are renewed. This includes pipe made from clear, air-dried redwood.

*Class B.*—This class includes coated pine or fir, in such a situation as to be open to continuous inspection, so that it may be given constant attention, comprising re-painting staves and renewing bands.

This pipe will be placed under Class *A*, on theory only, as experience has not yet confirmed such an assumption.

*Class C.*—This class will have a maximum life of 10 years and an average life of 7 years. It will include uncoated fir, pine, or other suitable wood.

### Machine-Banded Pipe.

*Class A.*—This class will have a life of from 15 to 25 years when receiving no attention; a longer life under ideal conditions, as when laid in soils having the least possible corrosive effect on the galvanized wire, and when operating under pressure, so as to insure complete saturation of the wood. Pipes of this class will be guaranteed to

withstand severe conditions of over-load, such as in hydro-electric work, general water-works for city supply, and high-pressure pumping lines; and will be guaranteed to withstand an over-load of 100% under test.

It will include pipes of clear, air-dried, redwood, manufactured according to the specifications in the Appendix.

*Class B.*—This class will have a life of at least 10 years, and a probable life not exceeding 15 years. It will include pipe made of redwood, or of coated fir or pine, etc., manufactured according to present-day standards, as indicated by the specifications covering this class.

*Class C.*—Pipes of this class will be used for temporary work only, and may be manufactured from redwood, fir, pine, or any other wood, with or without coating, as desired.

## APPENDIX.

### SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *A*.

The staves shall be of clear, air-dried, California redwood, seasoned at least one year in the open air, and shall be free from knots (except small knots appearing on one face only), sap, dry rot, wind-shakes, pitch, pitch seams, pitch pockets, or other defects which would materially impair their strength or durability. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe, and the edges shall be beveled to true radial planes. The staves shall be milled from stock sizes of lumber, the net finished thickness of the stave, for the various diameters of pipe, shall be as given in Table 2. The ends shall be cut square and slotted to receive the metallic tongues which form the butt joints. The slots shall appear in the same position on each stave, and shall be cut to make a tight fit with the tongues in all directions. The staves shall have an average length of at least 15 ft. 6 in., and not more than 1% shall have a length of less than 9 ft. 6 in. Staves shorter than 8 ft. will not be accepted.

The metallic tongues to insert in the slots in the ends of the staves shall be made from $1\frac{1}{2}$ by $\frac{1}{8}$-in. band iron, and shall be cut $\frac{1}{16}$ in. longer than the slot in the stave, so that, after the pipe is cinched, they will penetrate the adjoining staves, thereby making a water-tight joint.

The bands for pipes of large diameter shall be in two sections; those for the smaller sizes shall be in one section. The bands shall be spaced on the pipe with a factor of safety of at least four, and shall consist of round, mild-steel rods, connected with malleable-iron shoes. Either open-hearth or Bessemer steel may be used. The phosphorous content in open-hearth steel shall not exceed 0.06; in Bessemer steel it shall not exceed 0.10. The ultimate strength shall be from 55 000 to 65 000 lb. per sq. in. Steel having an ultimate strength of more than 65 000 lb. per sq. in. will not be rejected provided it shows an elongation of not less than 26% in 8 in. The yield points shall be not less than one-half the ultimate strength, and shall be determined by the drop of the beam of the testing machine. A minimum percentage in 8 in. of 1 400 000 divided by the ultimate tensile strength shall be taken as the elongation; but, the following modifications shall be made for bands less than $\frac{7}{16}$ in. and more than $\frac{3}{4}$ in. in diameter.

(*a*). For each increase of $\frac{1}{8}$ in. in diameter greater than $\frac{3}{4}$ in. a deduction of 1 shall be made from the specified percentage of elongation.

(*b*). For each decrease of $\frac{1}{16}$ in. in diameter less than $\frac{7}{16}$ in. a deduction of 1 shall be made from the specified percentage of elongation.

The rods or bands shall be capable of bending 180° around a diameter equal to that of the specimen tested, without fracture on either side. The threads shall be cold-rolled, United States Standard; the threaded portion of the band shall have an ultimate strength equal to that required for the rods. The nut shall conform to the Colorado Fuel and Iron Company's standard* for the respective diameters, and shall be tapped so as to make a snug but easy running fit. The bands shall be provided with button heads, according to the Colorado Fuel and Iron Company's standard,* and the heads and the sections under the heads shall not fail at less than is required for the body of the rod when tested through a U-slot. One bending and two tension tests shall be made on the rods for each melt of open-hearth steel, and one bending and one tension test for each blow of Bessemer steel rolled. Two bands for each blow or melt shall be tested against the head and thread. The bands shall be subject to rejection if the actual weight of any lot varies more than 5% above or below the theoretical weight of that lot. The bands shall be free from any injurious seams, flaws, or cracks, and shall have a workmanlike finish.

The shoes shall be of the Allen type, fitting closely to the outside curvature of the pipe, and designed so that, after the bands are cinched tight, they will lie in a plane at right angles to the horizontal axis of the pipe. The shoes shall be clean castings, made from the best grade of malleable iron, free from flaws, tags, or blow-holes; and shall have a tensile strength of about 40 000 lb. per sq. in. The shoes shall be guaranteed to be stronger under test than the bands with which they are to be used.

The coating for all metal work—shoes, bands, or tongues—shall be a high-grade preservative paint, of such consistency that it will not run in hot weather or peel off in cold weather.

The coating for the bands shall be hot, and the bands shall remain in the liquid for sufficient time to insure that they will attain the same temperature as the liquid.

When a conduit is to be erected in a tropical climate, similar to that of the Hawaiian Islands, the Philippines, Central America, etc., all metal work shall be protected with red lead or galvanizing. The red lead shall be of the best quality, containing approximately 10% of litharge to insure drying without scaling. The bands, on coming from the rolls, shall be dipped in linseed oil to prevent the formation of mill scale prior to the application of the red lead. The galvanizing shall be of a standard quality, giving a full and complete coating to the metal over its entire surface.

The diameters of the rods and the maximum allowable spacing shall be as given in Table 2.

* Or other specified standard—the C. F. & I. being that generally accepted.

## TABLE 2.—Details of Design for Continuous-Stave Pipe. Classes *A*, *B*, and *C*.

| Size of pipe, in inches. | Stave Thickness, in Inches. Standard. | Stave Thickness, in Inches. Maximum. | Number of Staves to Circle. Standard. | Number of Staves to Circle. Maximum. | Stock Size of Lumber, in Inches. Standard. | Stock Size of Lumber, in Inches. Maximum. | Number of Feet. Board Measure, per Foot of Pipe. Standard. | Number of Feet. Board Measure, per Foot of Pipe. Maximum. | Top Width of Stave, in Inches. Standard. | Top Width of Stave, in Inches. Maximum. | Size of band, in inches. | Number of pieces in band. | Spacing of Bands, in Inches. Maximum allowable. | Spacing of Bands, in Inches. For 100-ft. head. | Radius of curvature to which pipe can be built, in feet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 1 3/8 | 1 5/8 | 13 | 13 | 2×4 | 2×4 | 8 2/3 | 8 2/3 | 3.560 | 3.679 | 3/8 | 1 | 10 | 6.38 | 60 |
| 14 | 1 3/8 | 1 11/16 | 15 | 15 | 2×4 | 2×4 | 10 | 10 | 3.512 | 3.642 | 3/8 | 1 | 10 | 5.45 | 70 |
| 16 | 1 7/16 | 1 11/16 | 17 | 17 | 2×4 | 2×4 | 11 1/3 | 11 1/3 | 3.496 | 3.588 | 3/8 | 1 | 10 | 4.76 | 80 |
| 18 | 1 7/16 | 1 11/16 | 18 | 19 | 2×4 | 2×4 | 12 | 12 2/3 | 3.659 | 3.545 | 7/16 | 1 | 10 | 5.76 | 90 |
| 20 | 1 7/16 | 1 11/16 | 20 | 20 | 2×4 | 2×4 | 13 1/3 | 13 1/3 | 3.617 | 3.688 | 7/16 | 1 | 10 | 5.20 | 100 |
| 22 | 1 7/16 | 1 11/16 | 22 | 22 | 2×4 | 2×4 | 14 2/3 | 14 2/3 | 3.572 | 3.643 | 7/16 | 1 | 10 | 4.73 | 110 |
| 24 | 1 7/16 | 1 11/16 | 23 | 24 | 2×4 | 2×4 | 15 1/3 | 16 | 3.695 | 3.604 | 7/16 | 1 | 10 | 4.34 | 120 |
| 26 | 1 1/2 | 1 9/16 | 17 | 17 | 2×6 | 2×6 | 17 | 17 | 5.374 | 5.397 | 7/16 | 1 | 10 | 4.00 | 130 |
| 28 | 1 1/2 | 1 9/16 | 18 | 18 | 2×6 | 2×6 | 18 | 18 | 5.432 | 5.453 | 7/16 | 1 | 10 | 3.72 | 140 |
| 30 | 1 1/2 | 1 9/16 | 19 | 19 | 2×6 | 2×6 | 19 | 19 | 5.477 | 5.498 | 1/2 | 1 | 10 | 4.53 | 150 |
| 32 | 1 1/2 | 1 9/16 | 20 | 20 | 2×6 | 2×6 | 20 | 20 | 5 525 | 5.545 | 1/2 | 1 | 10 | 4.25 | 160 |
| 34 | 1 1/2 | 1 9/16 | 21 | 21 | 2×6 | 2×6 | 21 | 21 | 5.562 | 5.580 | 1/2 | 1 | 10 | 3.98 | 170 |
| 36 | 1 9/16 | 1 5/8 | 22 | 22 | 2×6 | 2×6 | 22 | 22 | 5.620 | 5.638 | 1/2 | 1 | 10 | 3.77 | 180 |
| 38 | 1 9/16 | 1 5/8 | 23 | 23 | 2×6 | 2×6 | 23 | 23 | 5.657 | 5.674 | 1/2 | 1 | 10 | 3.57 | 190 |
| 40 | 1 9/16 | 1 5/8 | 24 | 25 | 2×6 | 2×6 | 24 | 25 | 5.682 | 5.471 | 1/2 | 1 | 10 | 3.39 | 200 |
| 42 | 1 5/8 | 1 11/16 | 26 | 26 | 2×6 | 2×6 | 26 | 26 | 5 513 | 5.516 | 1/2 | 1 | 10 | 3.23 | 210 |
| 44 | 1 5/8 | 1 11/16 | 27 | 27 | 2×6 | 2×6 | 27 | 27 | 5.536 | 5.551 | 1/2 | 1 | 10 | 3.09 | 220 |
| 46 | 1 5/8 | 1 11/16 | 28 | 28 | 2×6 | 2×6 | 28 | 28 | 5.570 | 5.580 | 1/2 | 1 | 10 | 2.96 | 230 |
| 48 | 1 5/8 | 1 11/16 | 29 | 29 | 2×6 | 2×6 | 29 | 29 | 5.596 | 5.615 | 1/2, or 5/8 | 1 | 10 | {2.84<br>4.41} | 240 |
| 50 | 1 5/8 | 1 11/16 | 30 | 30 | 2×6 | 2×6 | 30 | 30 | 5.618 | 5.632 | 5/8 | 1 | 10 | 4.24 | 250 |
| 52 | 1 5/8 | 1 11/16 | 31 | 31 | 2×6 | 2×6 | 31 | 31 | 5.652 | 5.662 | 5/8 | 1 | 10 | 4.07 | 260 |
| 54 | 2 1/2 | 2 11/16 | 33 | 34 | 3×6 | 3×6 | 49 1/2 | 51 | 5.658 | 5.528 | 5/8 | 2 | 10 | 3.92 | 270 |
| 56 | 2 1/2 | 2 11/16 | 34 | 35 | 3×6 | 3×6 | 51 | 52 1/2 | 5.678 | 5.552 | 5/8 | 2 | 10 | 3.80 | 280 |
| 58 | 2 1/2 | 2 11/16 | 36 | 36 | 3×6 | 3×6 | 54 | 54 | 5.542 | 5.574 | 5/8 | 2 | 10 | 3.66 | 290 |
| 60 | 2 1/2 | 2 11/16 | 37 | 37 | 3×6 | 3×6 | 55 1/2 | 55 1/2 | 5.563 | 5.595 | 5/8 | 2 | 10 | 3.54 | 300 |
| 62 | 2 1/2 | 2 11/16 | 38 | 38 | 3×6 | 3×6 | 57 | 57 | 5.584 | 5.615 | 5/8 | 2 | 10 | 3.42 | 310 |
| 64 | 2 1/2 | 2 11/16 | 39 | 39 | 3×6 | 3×6 | 58 1/2 | 58 1/2 | 5.604 | 5.634 | 5/8 | 2 | 10 | 3.32 | 320 |
| 66 | 2 1/2 | 2 11/16 | 40 | 40 | 3×6 | 3×6 | 60 | 60 | 5.622 | 5.562 | 5/8 | 2 | 10 | 3.22 | 330 |
| 68 | 2 1/2 | 2 11/16 | 41 | 41 | 3×6 | 3×6 | 61 1/2 | 61 1/2 | 5.640 | 5.669 | 5/8 | 2 | 10 | 3.12 | 340 |
| 70 | 2 1/2 | 2 11/16 | 42 | 42 | 3×6 | 3×6 | 63 | 63 | 5.657 | 5.685 | 5/8 | 2 | 10 | 3.03 | 350 |
| 72 | 3 1/2 | 3 11/16 | 44 | 45 | 4×6 | 4×6 | 88 | 90 | 5.685 | 5.585 | 5/8, 3/4 | 2 | 8, 10 | {2.95<br>4.24} | 430 |
| 74 | 3 1/2 | 3 11/16 | 46 | 46 | 4×6 | 4×6 | 92 | 92 | 5.579 | 5.604 | 3/4 | 2 | 10 | 4.12 | 440 |
| 76 | 3 1/2 | 3 11/16 | 47 | 47 | 4×6 | 4×6 | 94 | 94 | 5.594 | 5.620 | 3/4 | 2 | 10 | 4.02 | 450 |
| 78 | 3 1/2 | 3 11/16 | 48 | 48 | 4×6 | 4×6 | 96 | 96 | 5.610 | 5.634 | 3/4 | 2 | 10 | 3.92 | 470 |
| 80 | 3 1/2 | 3 11/16 | 49 | 49 | 4×6 | 4×6 | 98 | 98 | 5.625 | 5.649 | 3/4 | 2 | 10 | 3.81 | 480 |
| 82 | 3 1/2 | 3 11/16 | 50 | 50 | 4×6 | 4×6 | 100 | 100 | 5.640 | 5.663 | 3/4 | 2 | 10 | 3.72 | 490 |
| 84 | 3 1/2 | 3 11/16 | 51 | 51 | 4×6 | 4×6 | 102 | 102 | 5 654 | 5.677 | 3/4 | 2 | 10 | 3.63 | 500 |
| 86 | 3 1/2 | 3 11/16 | 52 | 53 | 4×6 | 4×6 | 104 | 106 | 5.667 | 5.583 | 3/4 | 2 | 10 | 3.55 | 516 |
| 88 | 3 1/2 | 3 11/16 | 53 | 54 | 4×6 | 4×6 | 106 | 108 | 5.681 | 5.596 | 3/4 | 2 | 10 | 3.48 | 528 |
| 90 | 3 1/2 | 3 11/16 | 55 | 55 | 4×6 | 4×6 | 110 | 110 | 5.588 | 5.609 | 3/4 | 2 | 10 | 3.39 | 540 |
| 92 | 3 1/2 | 3 11/16 | 56 | 56 | 4×6 | 4×6 | 112 | 112 | 5.602 | 5.623 | 3/4 | 2 | 10 | 3.32 | 552 |
| 94 | 3 1/2 | 3 11/16 | 57 | 57 | 4×6 | 4×6 | 114 | 114 | 5.616 | 5.637 | 3/4 | 2 | 10 | 3.25 | 564 |
| 96 | 3 5/8 | 3 11/16 | 58 | 58 | 4×6 | 4×6 | 116 | 116 | 5.642 | 5.649 | 3/4 | 2 | 8 | 3.18 | 575 |
| 108 | 3 5/8 | 3 11/16 | 65 | 65 | 4×6 | 4×6 | 130 | 130 | 5.620 | 5.626 | 3/4 | 2 | 8 | 2.83 | 750 |
| 120 | 3 5/8 | 3 11/16 | 71 | 72 | 4×6 | 4×6 | 142 | 144 | 5.681 | 5.614 | 3/4 | 2 | 8 | 2.54 | 1 080 |
| 132 | 3 5/8 | 3 11/16 | 78 | 78 | 4×6 | 4×6 | 156 | 156 | 5.659 | 5.669 | 3/4 | 2 | 8 | 2.32 | 1 190 |
| 144 | 3 5/8 | 3 3/4 | 85 | 85 | 4×6 | 4×6 | 170 | 170 | 5.642 | 5.651 | 3/4, 7/8 | 2 | 8, 6 | {2.12<br>2.89} | 1 300 |

SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *B*.

The staves for uncoated redwood pipe shall be the same as those specified for Class *A*, continuous-stave pipe.

The staves for coated pipe shall be of yellow fir (Douglas fir), redwood, or such other wood, acceptable to the engineer, as may be specified by the bidder at the time of submitting his proposal. The wood shall be sound, straight-grained, and free from dry rot, pitch seams, pitch pockets, checks, wind-shakes, bruised ends, sap wood, and other imperfections which would impair its strength or durability. Through knots or knots at the ends or edges of staves will not be allowed. Sound knots and knots not exceeding ½ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned, for not less than 60 days, by air-drying in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; and the edges shall be beveled to true radial planes. The ends shall be cut square and slotted to receive the metallic tongues which form the butt joints. The slots shall appear in the same position on each stave, and shall be cut to make a tight fit with the tongues in all directions. The staves shall have an average length of at least 16 ft., and not more than 1% shall have a length of less than 9 ft. 6 in. Staves shorter than 8 ft. will not be accepted. The specifications for the tongues, rods, and shoes, and for the coating of the metal work shall be the same as for Class *A* pipe.

Redwood pipe need not be protected with any artificial coating. Pipe made of fir or other wood shall be coated. This coating shall be continuous and heavy; it shall be not less than $\frac{1}{16}$ in. thick, and shall consist of more than one individual coat of a mixture of asphaltum and tar. The first coating shall be allowed to dry thoroughly before the application of the second. The coating shall be hard, tough, durable, perfectly water-proof, and strongly adhesive to the metal and the staves. It shall show no tendency to flow under a summer temperature, and shall not become brittle, so as to crack or scale, under a freezing temperature. The coating shall be well spread and rubbed in with brushes, or shall be applied as a spray under pressure; but, in either case, all cracks, checks, or other surface irregularities shall be thoroughly covered and filled.

SPECIFICATIONS FOR CONTINUOUS-STAVE PIPE, CLASS *C*.

The staves shall be of Douglas fir, redwood, or other wood acceptable to the engineer. It shall be sound, straight-grained, free from dry rot, checks, wind-shakes, and other imperfections which would impair its strength or adaptability for pipe construction. Sap will

not be allowed on more than 10% of the inside face of any stave, and in not more than 10% of the total number of pieces. The sap shall be bright, and shall not occur within 4 in. of the end of any piece. Pitch seams will be permitted in not more than 10% of the total number of pieces, if showing on the edge only and if not longer than 4 in. or wider than $\frac{1}{16}$ in. Through knot or knots at the edge or within 6 in. of the ends of the staves will not be allowed. Sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe, and the edges shall be beveled to true radial planes.

The remainder of the specifications are as outlined for Class *B* pipe, except that the coating of the pipe may be omitted.

### Specifications for Machine-Banded Pipe, Class *A*.

The staves shall be of clear, air-dried, California redwood, seasoned at least one year in the open air, and shall be free from knots (except small knots appearing on one face only), sap, dry rot, windshakes, pitch, pitch seams, pitch pockets, or other defects which would materially impair their strength or durability. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe, the edges shall be beveled to true radial planes, and shall also have a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 20 ft. The staves shall be milled from stock sizes of lumber. The net finished thickness of the stave, for the various diameters of pipe, shall be as given in Table 3.

Both pipe and wire-wound collars, when such are used, shall be wound spirally with a heavily galvanized steel pipe-winding wire. This wire shall be of such a size, and spaced at such a distance (according to the head under which the pipe will operate), as to give a factor of safety of at least four against breaking. This wire shall also be of such a size, and spaced at such a distance as to give a bearing surface which will make the pipe safe against failure by the wire sinking into the wood under pressure, which might cause the pipe to leak along the longitudinal joints. The sizes and spacing of the wire for the various sizes of pipe operating under different pressure heads shall be as given in Table 3.

The ends of the wire shall be fastened securely to the pipe with pressed-steel or malleable-iron clips, which shall be protected against

TABLE 3.—MACHINE-BANDED PIPE.

| Diameter of pipe, in inches. | Thickness of staves, in inches. | No. of wire gauge. | SPACING OF WIRE, IN INCHES, FOR VARIOUS HEADS, IN FEET. | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 25 | 50 | 75 | 100 | 125 | 150 | 175 | 200 | 225 | 250 | 275 | 300 | 325 | 350 | 375 | 400 |
| CLASS *A*. | | | | | | | | | | | | | | | | | | |
| 2 | 7/8 | 12 | 3 1/2 | 3 1/2 | 2 7/16 | 1 13/16 | 1 7/16 | 1 3/16 | 1 1/16 | 15/16 | 13/16 | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 |
| 3 | 7/8 | 12 | 3 1/2 | 3 1/16 | 2 1/16 | 1 3/8 | 1 1/4 | 1 | 7/8 | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 | 7/16 | 3/8 |
| 4 | 7/8 | 12 | 3 1/2 | 2 3/4 | 1 7/8 | 1 3/8 | 1 1/8 | 15/16 | 13/16 | 11/16 | 5/8 | 9/16 | 1/2 | 7/16 | 7/16 | 3/8 | 3/8 | 5/16 |
| 5 | 7/8 | 12 | 3 1/2 | 2 3/8 | 1 5/8 | 1 3/16 | 15/16 | 13/16 | 11/16 | 5/8 | 1/2 | 1/2 | 7/16 | 3/8 | 3/8 | 5/16 | 5/16 | 5/16 |
| 6 | 7/8 | 10 | 3 1/2 | 3 1/4 | 2 1/8 | 1 5/8 | 1 1/4 | 1 1/16 | 15/16 | 13/16 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 | 7/16 | 3/8 |
| 8 | 15/16 | 8 | 3 1/2 | 3 1/2 | 2 7/16 | 1 13/16 | 1 7/16 | 1 1/4 | 1 1/16 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 |
| 10 | 15/16 | 8 | 3 1/2 | 2 13/16 | 1 7/8 | 1 7/16 | 1 1/8 | 15/16 | 13/16 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 | 3/8 | 3/8 | 5/16 |
| 12 | 15/16 | 8 | 3 1/2 | 2 3/8 | 1 1/2 | 1 3/16 | 15/16 | 3/4 | 11/16 | 9/16 | 1/2 | 7/16 | 7/16 | 3/8 | 3/8 | 5/16 | 5/16 | 5/16 |
| 14 | 1 1/8 | 8 | 3 1/2 | 2 | 1 5/16 | 1 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 6 | ..... | ..... | ..... | ..... | 1 1/8 | 15/16 | 13/16 | 11/16 | 5/8 | 9/16 | 1/2 | 7/16 | 7/16 | 3/8 | 3/8 | 5/16 |
| 16 | 1 1/8 | 6 | 3 1/2 | 2 1/8 | 1 1/8 | 1 1/4 | 1 | 13/16 | 11/16 | 5/8 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 4 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 7/16 | 7/16 | 7/16 |
| 18 | 1 1/8 | 6 | 3 1/2 | 2 1/8 | 1 3/8 | 1 1/16 | 7/8 | 11/16 | 5/8 | 1/2 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 4 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 | 3/8 | 3/8 | 3/8 |
| 20 | 1 1/8 | 6 | 3 1/2 | 2 | 1 1/8 | 1 | 3/4 | 5/8 | 9/16 | 1/2 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 4 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | 9/16 | 1/2 | 1/2 | 7/16 | 3/8 | 3/8 | 3/8 | 5/16 |
| 22 | 1 1/8 | 6 | 3 1/2 | 1 3/4 | 1 3/16 | 7/8 | 11/16 | 9/16 | 1/2 | 7/16 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 4 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | 1/2 | 1/2 | 7/16 | 3/8 | 3/8 | 5/16 | 5/16 | 5/16 |
| 24 | 1 1/8 | 6 | 3 | 1 5/8 | 1 | 13/16 | 5/8 | 1/2 | 7/16 | 3/8 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| | | 4 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | 1/2 | 7/16 | 3/8 | 3/8 | 3/8 | 5/16 | 5/16 | 1/4 |
| CLASSES *B* AND *C*. | | | | | | | | | | | | | | | | | | |
| 2 | 1 | 8 | 4 | 4 | 4 | 3 | 2 3/8 | 2 | 1 11/16 | 1 1/2 | 1 5/16 | 1 3/16 | 1 1/16 | 15/16 | 7/8 | 13/16 | 3/4 | 11/16 |
| 3 | 1 | 8 | 4 | 4 | 3 5/16 | 2 1/2 | 2 | 1 5/8 | 1 3/8 | 1 1/4 | 1 1/16 | 1 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 5/8 |
| 4 | 1 1/8 | 6 | 4 | 4 | 3 5/8 | 2 3/4 | 2 1/4 | 1 13/16 | 1 9/16 | 1 3/8 | 1 3/16 | 1 1/16 | 1 | 7/8 | 13/16 | 3/4 | 11/16 | 11/16 |
| 5 | 1 1/8 | 6 | 4 | 4 | 3 7/16 | 2 9/16 | 2 1/16 | 1 11/16 | 1 7/16 | 1 1/4 | 1 1/8 | 1 | 15/16 | 13/16 | 3/4 | 11/16 | 11/16 | 5/8 |
| 6 | 1 1/8 | 4 | 4 | 4 | 3 3/4 | 2 13/16 | 2 1/4 | 1 7/8 | 1 5/8 | 1 3/8 | 1 1/4 | 1 1/8 | 1 | 15/16 | 7/8 | 13/16 | 3/4 | 3/4 |
| 8 | 1 1/4 | 4 | 4 | 4 | 3 9/16 | 2 11/16 | 2 3/16 | 1 3/4 | 1 1/2 | 1 5/16 | 1 3/16 | 1 1/16 | 15/16 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 |
| 10 | 1 1/4 | 4 | 4 | 4 | 3 7/16 | 2 9/16 | 2 1/16 | 1 11/16 | 1 7/16 | 1 1/4 | 1 1/8 | 1 | 15/16 | 13/16 | 3/4 | 11/16 | 11/16 | 5/8 |
| 12 | 1 1/4 | 2 | 4 | 4 | 3 7/8 | 2 15/16 | 2 5/16 | 1 15/16 | 1 5/8 | 1 7/16 | 1 1/4 | 1 1/8 | 1 1/16 | 15/16 | 7/8 | 13/16 | 3/4 | 11/16 |
| 14 | 1 3/8 | 2 | 4 | 4 | 3 9/16 | 2 11/16 | 2 1/8 | 1 3/4 | 1 1/2 | 1 5/16 | 1 3/16 | 1 1/16 | 1 | 7/8 | 13/16 | 3/4 | 11/16 | 11/16 |
| 16 | 1 3/8 | 1 | 4 | 4 | 3 5/8 | 2 3/4 | 2 3/16 | 1 13/16 | 1 9/16 | 1 3/8 | 1 3/16 | 1 1/16 | 1 | 15/16 | 13/16 | 3/4 | 3/4 | 11/16 |
| 18 | 1 3/8 | 1 | 4 | 4 | 3 1/4 | 2 7/16 | 1 15/16 | 1 5/8 | 1 3/8 | 1 3/16 | 1 1/16 | 15/16 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 5/8 |
| 20 | 1 1/2 | 1 | 4 | 4 | 2 7/8 | 2 3/16 | 1 3/4 | 1 1/2 | 1 1/4 | 1 1/16 | 15/16 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 9/16 | 9/16 |
| 22 | 1 1/2 | 1 | 4 | 4 | 2 5/8 | 2 | 1 9/16 | 1 5/16 | 1 1/8 | 1 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 |
| 24 | 1 1/2 | 1 | 4 | 3 5/8 | 2 7/16 | 1 13/16 | 1 1/2 | 1 3/16 | 1 | 7/8 | 13/16 | 3/4 | 11/16 | 5/8 | 9/16 | 1/2 | 1/2 | 7/16 |

corrosion by galvanizing or Sherardizing, and there shall be no splices in the wire on any section of pipe or collar. The wire shall have a tensile strength of from 60 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers. The tension under which the wire is wound on the pipe shall be between 25 000 and 30 000 lb. per sq. in. All wire used in the manufacture of machine-banded pipe shall withstand a test for condition of galvanizing; this test shall consist of three of the four immersions in the testing solution, specified as the Western Union test, which reads as follows:

"Method of Testing.—Samples of wire, previously cleaned with gasoline or benzine, shall be immersed to a distance of at least 4 in.

in a glass vessel containing not less than one pint of the standard solution, and allowed to remain for one minute. They shall then be removed, washed in clear water, and wiped dry with soft cotton cloth or waste. This process shall be repeated three times, making four immersions in all."

A saturated solution of sulphate of copper, having a specific gravity of 1.186 and a temperature of 65° Fahr., shall be taken as the standard solution. The temperature of the solution during the test shall not be above 68° nor below 62° Fahr. If a bright copper deposit appears on the steel after the fourth immersion, thus indicating that the wire is exposed, the galvanizing represented by the samples shall be considered faulty. Three of these immersions without showing signs of copper, shall be considered as the test for the pipe winding wire.

Wooden collars or other couplings shall be furnished under the following specifications:

Continuous-stave and machine-banded collars shall be made in the same manner as the pipe, the staves being 6 or 8 in. long, depending on the diameter of the pipe. Pipe from 2 to 6 in. in diameter, inclusive, shall have collars 6 in. long; pipes of larger diameters shall have collars 8 in. long. The continuous-stave collar shall be banded with $\frac{3}{8}$-in. round, mild-steel rods, held together by straight-pull, malleable pipe shoes, and a nut. The machine-banded collar shall be wound with the same wire as the pipe, and in the same manner, but the wire shall be spaced closer in order to make it stronger than the pipe.

For the inserted joint connection, each section of pipe shall be mortised and tenoned with a male and female joint.

Inserted joint connections shall be used on pipe operating under pressures not exceeding a static head of 25 ft. and in sizes up to and including pipe of 12 in. inside diameter. All other sizes, and the aforementioned sizes under higher heads, shall have machine-banded collars when operating under static water pressure, or under a pumping pressure in which there will be no excessive pulsations. In all cases, machine-banded collars shall be preferred to the inserted or slip joint. On pipes having an inside diameter of 12 in. or greater, in pumping lines for hydro-electric work, where over-load, pulsation, or hammer are very likely to occur, continuous-stave collars shall be used.

All pipe shall be guaranteed to withstand an over-load of at least 50% when operating in flow lines not subject to over-load strains of any kind, and shall be guaranteed to withstand a 100% over-load when operating under all other conditions.

### Specifications for Machine-Banded Pipe, Class *B*.

The staves shall be of clear, air-dried redwood, uncoated, or of fir protected by a coating having an asphaltic base, and rolled in saw-dust.

The staves for uncoated redwood pipe shall be the same as specified for Class *A* machine-banded pipe.

The staves for coated pipe shall be of yellow fir (Douglas fir), redwood, or such other wood, acceptable to the engineer, as may be specified by the bidder at the time of submitting his proposal. The wood shall be sound, straight-grained, and free from dry rot, pitch seams, pitch pockets, checks, wind-shakes, bruised ends, sap wood, and other imperfections which would impair its strength or durability. Through knots or knots at ends or edges of staves will not be allowed. Sound knots and knots not exceeding ½ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of the staves shall be milled to conform to the inside and outside radii of the pipe; the edges shall be beveled to true radial planes, and shall be provided with a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 24 ft., the limits for redwood pipe being from 8 to 20 ft. and for fir pipe from 8 to 24 ft.

The thickness of the staves of redwood pipe shall be the same as specified for Class *A* pipe.

For pipe of fir and other woods the thickness of the staves shall be as given in Table 3.

The pipe shall be wound spirally with a special heavily galvanized, steel pipe-winding wire, and spaced with a factor of safety of four. The size of the wire shall depend on the diameter of the pipe and the pressure. Further, the wire shall be of such a size, and spaced at such a distance as to insure the pipe against failure by the wire sinking into the wood and allowing the longitudinal seams to open.

The ends of the wire shall be fastened securely with pressed-steel or malleable-iron clips, which shall be protected against corrosion by galvanizing or Sherardizing. The wire shall have a tensile strength of from 50 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers.

Redwood pipe may be furnished with corrections of the same type as specified for Class *A* pipe. If fir or other woods are used, cast-iron collars shall be provided for connecting sections of pipe. These cast-iron collars shall be of pure, gray iron, of the highest grade, free from tags, blow-holes, or other imperfections which would impair their strength. The inserted joint connections may be used under the same conditions as specified for Class *A* pipe, but shall be heavily

coated with protective compound after erection. If wire-wound collars or continuous-stave collars are used, they shall be made of redwood staves.

Redwood pipe need not be protected with any artificial coating. Pipe made of fir or other wood shall be coated. This coating shall be continuous and heavy; it shall be not less than $\frac{1}{16}$ in. thick, and shall consist of more than one individual coat of a mixture of asphaltum and tar. The coating shall be hard, tough, durable, perfectly water-proof, and strongly adhesive to the metal and the staves. It shall show no tendency to flow under a summer temperature, and shall not become brittle, so as to crack or scale, under a freezing temperature. The pipe shall be hot-dipped, and its tenoned ends shall be protected during the dipping so as to prevent the mixture from getting on the inside of the pipe. After the pipe has been dipped it shall be rolled down an incline covered with fine sawdust in order to cover it and enable it to be handled without the coating sticking to surfaces with which it comes in contact. The guaranties shall be the same as those applying to Class *A* pipe.

### Specifications for Machine-Banded Pipe, Class *C*.

The staves shall be of Douglas fir, redwood, or other wood acceptable to the engineer. The wood shall be sound, straight-grained, and free from dry rot, checks, wind-shakes, wane, and other imperfections which would impair its strength or adaptability for pipe construction. Sap will not be allowed on more than 10% of the inside face of any stave, and in not more than 10% of the total number of pieces. The sap shall be bright, and shall not occur within 4 in. of the end of any piece. Pitch seams will be permitted in not more than 10% of the total number of pieces, if showing on the edge only and if not longer than 4 in. or wider than $\frac{1}{16}$ in. Through knots or knots at the edge or within 6 in. of the ends of the staves will not be allowed. Sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the foregoing limitations, nor exceeding three within a 10-ft. length, will be accepted. Before milling, all lumber shall be seasoned by air-drying for not less than 60 days, in open piles, or by thorough kiln-drying. The sides of staves shall be milled to conform to the inside and outside radii of the pipe; the edges shall be beveled to true radial planes, and shall be provided with a small tongue and groove. After the staves are built up in sections, the ends shall be cut square, and a smooth tenon shall be turned on the end of each section, to make a tight fit with the collars or couplings. The sections shall be in random lengths of from 8 to 24 ft.

Both pipe and wire-wound collars, when such are used, shall be wound spirally with a heavily galvanized steel pipe-winding wire, and spaced with a factor of safety of not less than four. The wire shall

**also be spaced for each diameter so as to insure the pipe against failure by the wire sinking into the wood and allowing the longitudinal seams to open.**

**The ends of the wire shall be fastened securely with pressed-steel clips. The wire shall have a tensile strength of from 60 000 to 65 000 lb. per sq. in., and an elastic limit of not less than 50% of the ultimate strength, and shall be wound under sufficient tension to be firmly seated in the wood without crushing the fibers.**

**Inserted joint connections may be used for a pressure of 100 ft. or less; on higher pressures wire-bound collars shall be used. All pipe shall be guaranteed to withstand a 50% over-load of the pressure for which it is designed.**

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852


# PAPERS AND DISCUSSIONS



# CONTROL OF THE COLORADO RIVER AS RELATED TO THE PROTECTION OF IMPERIAL VALLEY

## Discussion.*


**By J. C. Allison, Assoc. M. Am. Soc. C. E.†**


**J. C. Allison,‡ Assoc. M. Am. Soc. C. E. (by letter).§—A statement of developments, in the control work of the Colorado River protecting the Imperial Valley, since the paper was written in October, 1915, and since the discussion was published in September, 1916, is of value.** **Mr. Allison.**

**During January, 1916, there occurred the greatest flood ever recorded on the Lower Colorado. At Yuma the discharge was 210 000 sec-ft., or 60 000 sec-ft. in excess of any previous measurement. The restricted channel opposite the Imperial Valley intake carried the flood, without side-scouring and with no meandering of the stream. The channel in the vicinity of "Houses 5 and 6" continued to widen and meander to the extent of undermining the primary line of defense. Bank erosion through the restricted area was not increased in proportion to the stream-bed erosion by the increase in velocity. The writer's statement that "the less room the river is allowed for meandering, the greater its velocity and scouring action, and the easier it is to keep it under control", to which Mr. Ockerson‖ has taken exception, is not only qualified here, but was again proved during the summer flood of 1916. The theory that the greater energy due to increasing velocity is expended on the bed of a stream, and not on the bank, is correct,**

*** Discussion of the paper by J. C. Allison, Assoc. M. Am. Soc. C. E., continued from September, 1916, *Proceedings*.**

**† Author's closure.**

**‡ Calexico, Cal.**


**§ Received by the Secretary, March 27th, 1917.**


**‖ *Proceedings*, Am. Soc. C. E., September, 1916.**

Mr. Allison. applied, of course, to locations where the stream alignment is good and where local deflections do not occur. Local deflections will not occur where the stream bed is narrow and deep, but they do occur where it is wide and shallow and the formation of shifting sand-bars is made easily possible.

At the breach in the levees between "Houses 5 and 6", which took place in January, 1916, the Inter-California Railroad, the second line of defense, held the flood-waters, and kept them from entering the Alamo Channel near the site of the original 1906 break. The breach occurred again from lack of maintenance, and the incident does not alter the advisability of retaining this line of defense and the present method of revetment. Where the breach occurred, the levees were designed with land-side borrow-pits, and though the flood-water through the breach entered these land-side borrow-pits, it was easily controlled.

On the Lower Colorado River protective work, past experience demonstrates beyond a doubt that the land-side borrow-pits are more advisable. The writer has even gone so far as to propose in one instance the construction of a drain canal in the borrow-pit parallel with the protective levee, for the purpose of preventing the saturation of the lands adjacent to the levees. In no instance has a levee failure been traced to the presence of a continuous pit on the land side, so long as the width of the levee and the width of the berm are sufficient.

The irrigation system and the system of protective work are now in the hands of the Imperial Irrigation District. Improvements to the flood-protection system of the Valley have been begun, the greater part of which are in accordance with the recommendations set forth in the paper. An additional control-gate is now being constructed above the present Hanlon gate, as suggested by Mr. Sonderegger.* The abandonment of the southerly end of the Hanlon Head-gate and the construction of additional openings in the rock to the north, are being planned to remedy the defects in that structure, as mentioned in the discussion.

The second line of levee-protection defense has been improved by placing a railroad track thereon throughout the whole extent of the Volcano Lake Levee, in accordance with early suggestions, and it is planned to open a quarry, in addition to the Hanlon Heading quarry, at the Volcano Lake end of the system. It is likewise planned to supply the district with sufficient equipment to facilitate present methods in maintaining the levee, namely, with rock revetment. In this regard, the suggestion made by Mr. Ockerson, which was carried out most efficiently and economically in his reconstruction of the Volcano Lake Levee, namely, that the rock facing should be laid by hand with some care, instead of being dumped at random from cars,

* *Proceedings*, Am. Soc. C. E., September, 1916.

is well founded in localities where the levee is not subjected to undermining from the river. This method of revetment is satisfactory for the entire second line of defense for Imperial Valley, but it is not practical on the primary line of defense and its extension, as proposed in the paper, on account of the difficulty of placing such rock to scour-line elevation. Mr. Allison.

The following is a résumé of the arguments and discussions concerning the theory and practices incident to the construction of a hydraulic-fill dam, as a means of diverting a stream flowing in an alluvial plain, which forms the principal subject of the paper.

The writer has made the statements that: "a direct relation exists between the character of the materials forming the bed and the velocities of the water", that "in refilling the bed, the materials are graded as to weight and deposited in direct relation to the velocities", and that in refilling the bed, the materials decrease "in weight as they approach the elevation corresponding with the lesser velocities." Mr. Ockerson cannot agree with this theory, and cites, as an example, the Mississippi River, in which bars composed of gravel larger than shingle exist well above the lower water and deep concave bends opposite the bars contain no gravel whatever. Mr. Ockerson's citation is of a stream in which the channel is permitted to deflect from one side to the other, and in which increased areas are developed by side scour rather than by bed scour.

Such a stream as that cited by Mr. Ockerson is that section of the Colorado River at the site of the proposed dam to deflect the Colorado River water from its present course into the Bee River, back into the old channel of the Colorado. It is on account of such a condition that the writer has proposed the construction of a hydraulic-fill dam up stream some 12 miles, where the channel has been virtually confined to one bed for a great number of years, rather than where it has been shifting in different locations each year. The Mississippi River condition and that which makes possible the construction of the hydraulic-fill dam in the Colorado River at Hanlon Heading and Yuma, are in no wise similar. At Yuma, scour is not influenced by bed-rock, as stated by Mr. Ockerson, because bed-rock at this point is at a considerably lower elevation than the bed-scour line, and a sample of the materials deposited in the bed will reveal that they are graded as to weight, increasing as the scour line is approached.

The construction of a temporary dam in the Colorado River at Hanlon Heading became necessary once more in the summer of 1916. The Irrigation District constructed a rock-fill dam rather than a hydraulic-fill dam, similar to that built the year before, on account of the lack of adequate equipment to permit of the latter type, and because of the greater river discharge at the time the dam became necessary.

Mr. Allison. The river discharges throughout the period when the dam was required exceeded 30 000 sec-ft. The writer is of the opinion that with the proper equipment, as outlined in the discussion of the paper, the hydraulic-fill dam might have been placed successfully, at least with sufficient success to have restricted the channel area and to have developed the necessary head for diversion of sufficient water into the Imperial Valley Canal. As it was, a rock-fill dam was again placed across the Colorado River and later removed, at a total cost of more than $100 000.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## TUNNEL WORK ON SECTIONS 8, 9, 10, AND 11, BROADWAY-LEXINGTON AVENUE SUBWAY NEW YORY CITY

### Discussion.*

By Israel V. Werbin, Assoc. M. Am. Soc. C. E.†

Israel V. Werbin,‡ Assoc. M. Am. Soc. C. E. (by letter).§—Mr. Moulton's discussion has perhaps set many to thinking of the possibility of adapting mining methods to excavating tunnels for engineering construction. The suggestions made by him are undoubtedly interesting, but, as one who has been intimately connected with the work described, the writer must take exception to his conclusion that mining methods could have been used advantageously on certain parts of the work. Mr. Werbin.

Referring to the large double-deck tunnels, with a cross-section approximately 40 ft. square, Mr. Moulton states:

"The work on these sections presents problems more nearly comparable to metal mining than those encountered anywhere else on subway construction in New York City."

The mistake is made right at the start in assuming that there is any similarity between metal mining and the work described in the paper. There were several important factors connected with the work in question which eliminated any possibility of using any of the methods suggested by Mr. Moulton, and these were:

(1) That the work was done under narrow streets, in built-up sections of the largest city in the world;

(2) That the tunnels were driven dangerously near the rock line;

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* Discussion of the paper by Israel V. Werbin, Assoc. M. Am. Soc. C. E., continued from December, 1916, *Proceedings*.

† Author's closure.

‡ New York City.

§ Received by the Secretary, March 19th, 1917.

Mr. Werbin.

(3) That the rock was known to be of a dangerous character; and
(4) That the problem was not merely one of making an excavation, but of doing this so that there would be absolutely no settlement of the street surface, and, most important of all, of building a structure inside of the finished excavation.

Surely none of these problems is to be met in mining work, particularly if credence is to be given to the occasional newspaper reports of large areas adjacent to mining operations settling, on account of the work done beneath them.

Mr. Moulton criticizes the use of the top heading as being antiquated and expensive. This manner of attack is followed almost everywhere in tunnel operations in America. This fact may not perhaps make it the better method, but there were other important considerations which made it the only practical method to use on work of this kind.

The tunnels in question were driven in connection with subway work, and, from the point of view of economy of construction, it was desirable to keep the structure as near to the street surface as possible. The importance of this feature will be realized when it is noted that the cost of the excavation in such work is approximately 50% of the cost of the entire construction. It is also important, from the standpoint of operation, for the railroad at the stations to be as near to the street surface as practicable, so as to reduce to a minimum the height of the stairs, and to dispense with the use of elevators.

When the first subway in New York was built, a portion on upper Broadway was built in a deep tunnel, and access to the stations at 168th, 181st, and 191st Streets is obtained by using elevators. Experience shows that with the large amount of traffic handled by the city railroads, the use of elevators is not only an expensive feature of operation, but is highly undesirable. For this reason, on the new work in the double-deck tunnels on Lexington Avenue, the express stations were placed immediately beneath the local tracks, so that communication with the local stations could be had by short stairways instead of elevators. In order to maintain reasonable grades, it was necessary to keep the express tunnels a short distance below the local tracks. If a bottom heading had been used, good rock (not requiring any timbering) would have been encountered all along the line, except for a stretch of about 200 ft. at 74th Street. There were, however, several places along Lexington Avenue where old watercourses ran across the line of the subway structure, and at these places the surface of the rock was considerably below the roof line. The material directly above the rock at these points was a fine quicksand or alluvial earth, which, when encountered, was very difficult to hold and keep from being washed into the excavation.

Under such conditions, the use of a bottom heading would have been dangerous. The top heading had the advantage of acting in the nature of an exploration tunnel, so that the contractor was prepared in advance for the conditions with which he had to deal. When bad ground was encountered, it was taken care of by timbering, with or without lagging boards, as might be necessary; and, when the overlying material was once properly caught up and held, the element of danger was practically removed. Assuming that under such conditions the work was conducted with a bottom heading, it would mean that a very expensive system of timbering would have been necessary every time another lift was taken out, with the continuing possibility of the danger and damage that such an operation would have involved. Mr. Werbin.

It must be borne in mind that the structure for the local tracks had been completed in advance of the express tunnel excavation, this being necessary in order to maintain necessary progress in the work. The loss of any material between the local structure and the express tunnels, therefore, would involve damage to the completed local track structure, in addition to settlement in the buildings at the street surface. It might also be mentioned that the contractor was held absolutely liable for all damage to buildings that resulted from his operations, irrespective of the matter of negligence, so that a procedure which would reduce to a minimum the possibility of damage might ultimately prove the most economical, even though some other method might permit the removal of the excavation more economically. When these matters are considered, the writer is sure that no mistake was made in using a top heading.

Mr. Moulton claims that with the methods of timbering ordinarily used in metal mining, it would be possible to work more nearly to the pay line of the excavation. The writer fails to see how the method of timbering can in any way affect the lines of excavation. This matter is determined more or less by the character of the rock and the way in which the strata lie. The writer believes that the men engaged on such works as that described in the paper are fully as competent as mining men to carry the excavation to approximate neat lines. The method of cribbing up the entire excavation might serve to hold some of the rock in the roof from falling down, but, as the writer has tried to bring out in the paper, no attempt was made to hold up such rock by timbering, the consensus of opinion being that the better practice would be to remove all loose rock. This conclusion was reached after careful study, there being little doubt that any timbering that would have to be put in to hold such rock in place, would be far more expensive than the additional cost of removing such excess excavation. The rock along the work in question had two seams running almost at right angles to each other, with the apex pointing upward, so that the rock in the roof was more or less wedge-shaped,

Mr. Werbin. with the bottom of the wedge downward. The very dangerous condition is evident, and, when this rock persisted in working loose, even where timbering was placed, there was no doubt that it was good practice to remove these large masses of loose rock rather than attempt to hold them in place, where they would have been an ever-continuing menace to the lives of the men working in the tunnel.

Mr. Moulton has criticized the segmental type of timbering as being expensive. The answer to this is that the problem was not only one of making an excavation, but also of building a structure inside the completed excavation. The square-set system of timbering used in mining, described by him, may be all right in a mine, but would make the cost of the construction that had to follow the excavation almost prohibitive. The subway structure could not have been erected without shifting considerable timber, and no engineer engaged on such work will deny that, once bad rock is caught up, it is very poor and dangerous practice to disturb the timbering.

The almost universal use of the segmental set in bad ground was prompted by the fact that, after it had been placed, the structure could be built beneath it with very little moving of the timbering. Another great advantage in the use of segmental timber in this work was the fact that, almost throughout the work, water was encountered in great quantities, and, by concreting between the timbers and grouting, a temporary roof was made. The water was then concentrated and led off through the grout pipes, and in this way was prevented from washing away the fresh concrete in the finished structure until it had had a chance to set.

The use of the longitudinal beams mentioned in the paper was not advocated so much on account of permitting any economy in removing the excavation, but solely because, with such a system, the posts directly under the sets did not have to be moved every time an additional lift was taken out; and it afforded a good foundation for the posts irrespective of soil conditions. The beams and posting, therefore, could be placed where they would offer the minimum interference with the construction work that was to follow. The Lexington Avenue work was the first in which the large longitudinal beams were used to any extent, and the advantage of their use has been so marked that this system is being adopted on much of the more recent work.

Mr. Moulton has presented some data as to the estimated cost of making the excavation if mining methods had been adopted. He does not seem to have taken local conditions into consideration, and if, to the prices he gives, he adds the cost incurred by the contractor practically having no choice in the location and the number of shafts, the high cost that must be paid in the city for labor, plant sites, dumping privileges, superintendence and insurance costs, and the

increased cost due to the drilling and blasting methods required in a job of this kind, the total sum for doing the work would be found to be higher than he apparently thinks it would be. Mr. Werbin.

The writer is sure that, if Mr. Moulton had been familiar with the conditions peculiar to this work, he would not have advocated the use of square-set timbering, or conducting the excavation with a bottom heading. It would seem to the writer that the only place where such a method could have been considered would have been in connection with the tunnels of 100th and 102d Streets, where the spans were greater than 60 ft., provided very bad ground had been encountered. The large I-beams to hold up the roof in this part of the work were used by the contractor mainly so that he could continue to utilize his air shovel, but, as stated in the paper, this method of procedure was found to be very expensive, and of little advantage, so that the use of the air shovel was abandoned and the mucking was done by hand.

It was found possible to excavate the section for sufficient width to permit the construction of one track of the structure at a time without using any timbering whatever, except for occasional posting, and, after the structure for this one track was completed and the roof had been caught up, the excavation for the adjacent track was made.

It would seem to the writer that the method followed was better than to attempt to make the excavation wider and use timbering. The only objection to the piecemeal method of construction was the damage to the finished structure that resulted from carrying on the blasting adjacent to it, but this could have been avoided had the contractor taken proper precautions.

It might have been advantageous to use the methods suggested by Mr. Moulton provided the rock encountered would not have permitted the excavation for one track with practically no timbering.

**AMERICAN SOCIETY OF CIVIL ENGINEERS**

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# PAPERS AND DISCUSSIONS



## THE YALE BOWL

### Discussion.*

BY MESSRS. ALEXANDER S. LYNCH AND CHARLES A. FERRY.†

ALEXANDER S. LYNCH,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—Mr. Ferry states that Sherardized bolts were first used to fasten the seats to the concrete and then abandoned for the bronze bolts, as originally planned. He does not state why this change was made, other than that the Sherardized bolts were unsatisfactory. The writer would like to know why they were unsatisfactory. Mr. Lynch.

Mr. Atwood states that the greatest movement in the high walls was ½ in. A recent examination of the walls shows that the movement in one was less than ¼ in., and in the other it was ¾ in., making a noticeable and very unsightly crack. The writer would like to ask if the soil under both walls was the same, or if there was a difference in the design of the walls.

The final cost of the bowl as given by Mr. Atwood is $750 000. Does this include the cost of the 23 000 seats erected for the Harvard games of 1914 and 1916, and subsequently removed?

There seems to have been no apparent reason for applying Truscon floor hardener to the aisles. Mr. Atwood states that no wear is perceptible. Neither is any wear perceptible in the floors of the tunnels, although the traffic there is much greater. There was no more reason for applying Truscon hardener to the aisles of the Bowl than for applying it to the concrete sidewalk. The only effect now noticeable is that the hardener was improperly applied, leaving many of the blocks a dirty, rusty brown, unpleasant to look at, and spoiling the general soft gray tone produced by the use of lampblack.

* Discussion of the paper by Charles A. Ferry, M. Am. Soc. C. E., continued from February, 1917, *Proceedings*.

† Author's closure.

‡ West Haven, Conn.

§ Received by the Secretary, March 24th, 1917.

Mr. Lynch. After reading Mr. Atwood's discussion, one is led to wonder why Mr. Ferry should have submitted a paper on the Bowl. Mr. Atwood gives "much credit" to the Committee of Consulting Engineers, which assisted in getting the work "properly started", and to others, but nowhere does he mention Mr. Ferry, except to refer to a paper read by him before the Connecticut Society of Civil Engineers. The writer, like hundreds of other engineers throughout the country, was under the impression that Mr. Ferry had not only designed the Bowl, but that he had in truth created "something new under the sun." The writer still holds that belief.

Mr. Ferry. CHARLES A. FERRY,* M. AM. SOC. C. E. (by letter).†—In reference to the cost of the Bowl, Mr. Atwood states: "when finally completed the cost will be about $750 000, or approximately $12.30 per seat." He does not give the details of this estimate, further than to state the different pieces of work required to complete the structure. Of these, the plans for the "permanent toilets" and the "permanent fence" have not been finally settled, so far as the writer knows; consequently, the cost might be any sum that Mr. Atwood chose to assume. It is questionable whether the cost of a fence enclosing the Bowl should be charged to that structure any more than that of the land on which it is built.

The sum paid to the contractor was a little less than $448 000. The plans for the Bowl were radically changed after the work was well advanced, as mentioned by Mr. French, thereby necessitating much extra work, as well as extra prices for considerable work already under contract. This also included a large sum for grading, which, although desirable in the development of the plans for the athletic field, was not necessary in the construction of the Bowl itself. It also included the cost of the temporary dressing rooms and the erection and removal of the temporary toilets and extra temporary seats for the Yale-Harvard game of 1914. If the extra temporary seats built for the Yale-Harvard games are to be included in the cost of the Bowl, as apparently they have been in Mr. Atwood's estimate, then, in all fairness, their number should be included in computing the cost per seat. On this basis, that cost would be only about $9.70 per seat, even at the extravagant estimate of $750 000.

The writer is confident that if the Bowl had been built as originally planned by him, the cost of the completed structure, including that of permanent toilets, which was not included in the original estimate, would not have exceeded $450 000, or about $7.40 per seat, as the contractor's bid, based on the engineer's list of quantities, was lower than the writer's estimate made on the same basis.

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* New Haven, Conn.

† Received by the Secretary, March 23d, 1917.

This price compares favorably with that quoted by Mr. Hitt for the Tacoma Stadium, namely, $6.20, when the relative sizes of the two structures are taken into account, as the price per seat increases with the increase in size of the structure, regardless of the materials used. Mr. Ferry.

Judging from Fig. 1 illustrating Mr. Hitt's admirable discussion, benches are not provided in the Tacoma Stadium, the spectators sitting directly on the concrete, as in the Harvard and Princeton Stadia. The permanent benches for the Bowl cost about $0.93 per seat, including painting.

The cost of the Tacoma Stadium would seem to confirm the writer's opinion that large stands constructed in the manner used by him in building the Bowl are more economical than those built by any other method hitherto used.

The history of this structure well illustrates the truth of the old adage that "too many cooks spoil the broth." Only, in this case, the "broth" (Bowl) was not spoiled, but the "high cost of living" (building) was very largely increased.

The writer is exceedingly skeptical regarding the utility of Mr. Atwood's method of consolidating the embankment, as described by him. No levels were taken on the embankment at the time, and, as there were no near-by permanent objects with which to compare the height of the embankment, the measurement of the settlement was dependent on the imagination of the observer. To believe that a given quantity of water, discharged at the bottom of small holes, 2 or 3 ft. deep and about 8 ft. apart (assuming that such holes existed in the saturated sand after the pipes or bars were withdrawn), would have any more effect in consolidating material several feet below the bottom of the holes, than an equal quantity percolating uniformly through the full depth of the sand, requires more credulity than is possessed by the writer.

As for the effect on the upper portion of the embankment by jetting water down into it, the writer is positive that the result was just the opposite of that desired. The water injected into the embankment washed out the finer particles of sand and deposited them in flat, volcano-like cones around the pipe, and the coarser particles were left in as loose a condition as it was possible to make them by the boiling action of the water escaping upward around the pipe. Only about one-half the circumference of the Bowl was treated by this novel method of "consolidating" material; and the opinion that this work was not only useless, but worse, would appear to be substantiated by the fact that the benches which showed the greatest subsequent settlement were in the sections thus treated, and the points of least settlement were in those portions which did not receive the "water cure",

Mr. Ferry. although the latter were on the west side of the Bowl where the depth of fill, between tunnels, was greatest, the tunnels on the east side being largely in cut.

Mr. Atwood states that:

"In the concrete facing inside the Bowl, the horizontal arching action was not made use of, dependence being placed on the interior retaining wall to care for any possible tendency of the blocks to slide down hill."

The writer has never had any fears that the concrete blocks would slide down hill, particularly those near the foot of the incline, where the slope is only about 1 on 4, unless pushed down by those near the top, where it is about 1 on 2½. If, however, there ever should be any such tendency to slide, he has much more faith in the resistance due to the arch action than in that of the inner retaining wall, for the concrete steps abut against the thin lip of the gutter formed in the top of this wall, and, although the lip is reinforced, the pressure required to shear it off is, probably, only a small fraction of that which would be required to crush or buckle the concrete facing under arch action. At the present time, 2½ years after the laying of the concrete on the lower part of the slope, there are no indications that there has been any sliding of the blocks down hill.

The experience of the past 2 years indicates the wisdom of the Committee in abandoning the plan for an open concrete facing raised above the embankment, as described by Mr. French, and in re-adopting the plan for a tight facing laid directly on the earth, as recommended by the Advisory Engineer, Mr. Williams.

Although the material of which the embankment is composed is very porous and readily absorbs the water from an ordinary rainstorm, it does not take care of that from heavy downpours, such as are likely to occur several times a year. Thus far, there has not been much trouble from the gullying of the inner face of the embankment, as the runs between the seats are in contact with the ground, thus forming water-breaks every 30 in. Nevertheless, after a very heavy rainstorm, the care-takers are obliged to shovel sand from the runs and fill up the gullies which have formed between them. If gullies, 3 or 4 in. in depth, will form in one storm when there are water-breaks every 30 in., it is reasonable to suppose that serious ones, such as might undermine the concrete footings, would form in a period of a few years when the water had an unobstructed run of 150 ft., even though the slope was covered with a layer of cinders.

The principal point in controversy between the writer and the engineers consulted by the Committee previous to the adoption of the plans, was whether the material in the embankment could be consolidated so that there would be no settlement which would be

likely to cause injurious cracking of the concrete facing. The writer's 20 years' experience in back-filling sewer trenches and, particularly, in the construction of an embankment on which to build a slow sand filter, led him to believe that it was possible, by the use of water, supplemented by thorough rolling, to construct a stable one for the Bowl. For the filter foundation, the maximum depth of the fill was about 16 ft. The material composing the fill was a mixture of clayey hardpan and a very fine sand saturated with water—practically quicksand. These materials were mixed roughly by working both borrow-pits at the same time, but there was no special effort to secure a thorough mix by harrowing. No water was used, except such as was contained in the quicksand and from occasional rains. The material was leveled off in layers about 8 in. in thickness, and was thoroughly rolled with a grooved roller; a smooth roller was not used. The fill was completed late in the fall. Benches were established at various points, after which an embankment, about 18 in. high, was built around the edges of the fill, and the area was flooded with water, the pond being maintained throughout the winter. In the spring, the water was drawn off, the benches were tested, and it was found that the maximum settlement was about $\frac{1}{8}$ in., a little better result than was obtained at the Bowl, although, at the latter, the depth of fill was nearly twice as great as at the filter foundation. Mr. Ferry.

The argument advanced by Mr. French, that because "the bulk of the fill had been exposed to a winter of snow and rain, the actual advantage of the rolling does not seem to have been conclusively demonstrated," does not apply to the section of the embankment, about 300 ft. in length, in the vicinity of the proposed large gate-house.

A portion of this fill, that immediately back of the retaining wall, is more than 40 ft. in depth (the deepest fill on the work) and all of it, from foundation to promenade, was deposited in the 9 weeks between April 4th and June 6th. The material was consolidated only by the use of water and rolling, as specified for the remainder of the work, the assistance rendered by Nature, in the form of snow and rain, being negligible, as the water secured in this way was insignificant in quantity in comparison with that applied artificially. The average settlement of this portion of the embankment, in the 2 years following its completion, was less than $\frac{3}{8}$ in.

The writer believes that any one who observed the action of the moistened sand under the rollers would readily admit that they performed a very important part in the consolidation of the material. The height of the ridges formed by the grooved roller was noticeably less with each successive passage of that machine; and the use of the two kinds of roller, passing alternately, was much more effective than that of either a smooth or grooved one alone would have been.

Mr. Ferry. The Sherardized bolts, referred to by Mr. Lynch, were lag-screws between the threads of which was spun a flat wire spiral to serve as reinforcement in transmitting the thrust from the screw to the concrete.

These bolts were mechanically defective in two respects. The wire spiral was likely to become slightly bent or sprung, in which case, only one thread of the screw would have a bearing, thus reducing the effective strength of the fastening. The defect which gave the most trouble, however, was caused by the machine which fashioned the bolt. This machine, in gripping the rod, changed the shape of the shank of the bolt, making the cross-section slightly oval instead of round, and, in addition, it formed a slight ridge along the sides of the shank along the major axis. The result was that, when the bolt was being unscrewed, it acted as a wedge which split a spall, several inches long, from the face of the step, thereby releasing the bolt. The only way in which this could be avoided was to give the bolt one or two turns just after the concrete had taken its initial set, thus forming a hole large enough to permit the bolt to be turned. If it was not turned until after the concrete had become hard, the head twisted off.

To repair the damage, dove-tailed recesses were chiseled into the steps, and the bolts were re-set in mortar. Several hundred bolts had to be re-set in this way before the writer succeeded in getting an order to discontinue their use.

In order to insure the head of the bolt having a firm bearing on the standards, the bolts were set about ½ in. lower than the position they were expected to occupy finally. In spite of this precaution, however, the concrete broken off in unscrewing the bolts filled up the holes to such an extent that it was impossible, in many cases, to screw the bolts down again to a bearing, and it was necessary to drill out the holes and blow out the dust with a blast from an air pump, the threads of the screw interfering with the use of a spoon.

In the writer's opinion, whenever it is necessary to use screws for attaching fixtures to concrete it should be done by building the bolt into the concrete, rather than the reverse. If, however, it is desirable that the bolt shall be the movable piece, then the nut should be in an enclosing shell, thereby preventing the thread of the screw from coming into contact with the concrete. Several hundred bolts of this type were used, with entire satisfaction, for fastening the Holophane globes to the ceiling of the tunnels.

The difference in the movement of the two walls, to which Mr. Lynch calls attention, was undoubtedly due to a change in the design of the structure, rather than to any difference in the foundation, as that is of the same general character throughout the field.

The material at the field consisted of strata, of sand and gravel, of varying thicknesses. Among these were three or four layers of a very

fine, compact sand—material which would be quicksand if in contact with water. One of these strata, about 6 in. thick, occurred about $3\frac{1}{2}$ ft. below the base of the high retaining walls, and was uncovered in excavating for the foundation of the adjacent tunnels. The construction manager expressed grave fears that, unless the foundation of the wall was carried down through this stratum and built on the coarse sand beneath it, the wall would slip. As this stratum was about 20 ft. above the level of ground-water, and as no water could reach it from above after the embankment was concreted, there was no danger of the sand becoming "quick"; therefore the writer believed that, to support the wall, this material was equal to, if not better than, the coarse but less compact sand beneath it, and he strongly objected to the expenditure of the several hundred dollars necessary to extend the footing as proposed. In spite of his protest, however, the extra footing was built under one wall, the forms for the other wall having been erected and the steel placed in position before Mr. Atwood became connected with the work. This extra footing, under the outer edge of the base of the T-shaped wall, was built in the form of the frustum of a wedge, with the narrow edge down, thus reducing the effective bearing area of the sand about one-half, the sides of the wedge having comparatively little supporting value. This is the wall which Mr. Lynch refers to as having moved $\frac{3}{4}$ in. The companion wall, at the other end of the gate-house, which he says has moved less than $\frac{1}{4}$ in. is the same in all respects, except that it is 1 ft. higher, thereby sustaining that much additional pressure, but is built with a flat base, as planned originally. Mr. Ferry.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## TESTS OF CONCRETE SPECIMENS IN SEA WATER, AT BOSTON NAVY YARD

### Discussion.*


By Marshall W. Brown, Assoc. M. Am. Soc. C. E.


Marshall W. Brown,† Assoc. M. Am. Soc. C. E.—The discussion of this paper has drifted toward the failure of concrete structures, due to the action of sea water. The speaker believes that great attention should be given to the nature of the stone used in the concrete. Mr. Brown.

If it is assumed that a structure has been adequately designed and properly built to withstand the forces acting on it as a structure, then its life depends mainly on its ability to resist the constant and all-powerful force of sea water acting on it as a destructive agent.

This force is both chemical and mechanical. The chemical reaction, being promoted by the presence of moisture, is facilitated by the mechanical action of the water, which, carrying away particles that have been disintegrated, presents new surfaces to be acted on.

The chemical destruction of any particular mass of concrete is slow. The rate of the mechanical destruction varies greatly, depending on the intensity of the wave action and the nature of the surface on which that action takes place.

The rate at which concrete disintegrates increases with the intensity of the wave action, and with the increasing roughness of the surfaces exposed.

In any volume of concrete there are three materials which are acted on, namely, the cement, the sand, and the stone. As the cement which binds the whole mass together is the one material which is

* Discussion of the paper by R. E. Bakenhus, M. Am. Soc. C. E., continued from March, 1917, *Proceedings*.

† New York City.

Mr. Brown. most readily acted on chemically, its rate of destruction must be reduced to a minimum. This is done in a number of ways:

First.—By reducing the quantity of cement needed in a mass, that is, by proportioning the sizes of the sand and stone in such a way that the volume of the voids filled by the cement in the mass is as small as possible.

Second.—By using sand and stone which are in themselves chemically inert and impervious to moisture, and by placing the mixture of these materials in such a manner that the mass, as a whole, when set, is impervious to moisture.

This eleminates all chance of disintegration of the interior of the mass, and confines it to the surface exposed directly to the action of the water. Without moisture, chemical action cannot take place.

The first disintegration of the surface of concrete takes place very slowly, and is mainly chemical. The surface is hard and smooth, and the water has slight effect on it mechanically. Gradually, the cement is eaten away, and a close examination shows that the area of chemical action has been greatly increased around the particles of sand. These are washed away, and, as the action continues, the stone is exposed. After a time, the whole surface, exposed to the action of the water, appears to be one mass of stone surrounded by small seams of mortar composed of sand and cement.

Thus far, only the outer surface of the structure has been displaced, and the integrity of the whole structure thereafter depends mainly on the character of the stone used. On it comes the chemical action of the sea water, the intensive mechanical action of the waves dashing against it, as well as the effect of frost, all tending to loosen and tear each individual particle of stone from the cement which binds it to the structure.

For this reason the speaker believes that the character of the stone has much to do with the life of a concrete structure exposed to the action of sea water. The stone in the concrete should be chemically inert, insoluble, non-absorbent, and should present a surface to which the cement will adhere with the greatest possible strength.

As far as local structures are concerned, this would practically limit such stone to crushed trap rock, granite, or washed quartz gravel.

The speaker has seen many failures of concrete structures in sea water, and in each instance he believes it has been due to the use of shale rock, sandstone, limestone, and other rocks of a sedimentary nature.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

### Discussion.*

By S. A. Knowles, Esq.

S. A. Knowles,† Esq.—In reference to the speed at the Rogers Pass Tunnel, Mr. McIlwee, the original contractor, who was responsible for the methods and organization, had had similar contracts at the Roosevelt Drainage Tunnel, in Cripple Creek, the Laramie-Poudre Tunnel, and several others, in which he had been successful in maintaining a high average footage. The organization which was assigned at the Rogers Pass Tunnel had been with him on his previous undertakings, and thus their combined experience enabled him to obtain such desirable results. Mr. Knowles.

An essential factor in obtaining a high and uniform footage was the ventilation. The speaker had an opportunity at one time to test the ventilating system, which comprised 12 000 ft. of practically straight 18-in. steel pipe of No. 16 gauge. A No. 7 Root blower, running at 100 rev. per min., gave a displacement of 6 500 cu. ft., and the indicated horse-power required for either blowing in or exhausting this quantity of air per minute was about 20.

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* Discussion of the paper by A. C. Dennis, M. Am. Soc. C. E., continued from March, 1917, *Proceedings*.

† Sault Ste. Marie, Mich.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## THE WATER SUPPLY OF PARKERSBURG, W. VA.

### Discussion.*

BY MESSRS. ALEXANDER POTTER, PHILIP BURGESS, JAMES H. FUERTES, AND MORRIS KNOWLES AND J. D. STEVENSON.

ALEXANDER POTTER,† ASSOC. M. AM. SOC. C. E.‡—The speaker is in hearty accord with Messrs. Fuller and Fuertes as to the wisdom of changing the present method of determining and reporting *B. coli.* When the warning signal rings just as loudly at the approach of a cow, or a tramp, walking on the railroad track, as it does for the fast moving express train, one is likely to have contempt for the signal or demand its change. When the presence of *B. coli* may mean evidence of contact with anything, from the human intestine to the covering of a kernel of wheat, one of two things must necessarily happen: contempt for the warning, or unnecessary and unwarranted alarm. It is difficult to say which is the more damaging. Mr. Potter.

This unnecessary alarm is exhibiting itself in the laws proposed for enaction in certain of the State legislatures. Certain legislation existing or proposed in many of the States indicates clearly that the authors or the sponsors for such legislation lack the fundamental principles of sanitary science. These laws seek to prevent the entry of sewage, either crude or purified, into the waters of the State. Such a bill is now before the New Jersey Legislature, submitted under the very attractive title, "An Act to Prevent the Pollution of Sources of

* Discussion of the paper by William M. Hall, M. Am. Soc. C. E., continued from March, 1917, *Proceedings*.

† New York City.

‡ This is a continuation of Mr. Potter's discussion published in *Proceedings* for March, 1917, p. 475.

Mr. Potter. Potable Water Supply in This State." The principal provision is contained in Paragraph 2, which reads as follows:

"2. It shall be unlawful for any person or persons, private or municipal corporation, to discharge into any water which may constitute or form part of a source of potable water supply any raw or treated sewage, or the effluent from any sewage disposal works, after the date fixed by the State Board of Health when such acts shall cease. Notice to discontinue the depositing of such sewage or effluent shall be given by the State Board of Health to such person, or private or municipal corporation, at least three months before the date fixed by said State Board of Health as aforesaid."

Other provisions of the Act leave the State Board of Health no alternative but to enforce the law prohibiting the discharge of the effluent of any sewage disposal plant, no matter what its degree of purity may be.

Another bill, almost as drastic, has been made a law by the State Legislature of Texas, and, in Oklahoma, the Health Officer, 6 years ago, laid down the principle that neither crude sewage nor purified sewage would be permitted to enter the streams of the State. Even New York State is now seriously contemplating the abandonment of hundreds of thousands of dollars spent on new public institutions, because the authorities feel that they cannot even trust the engineer to design a plant which will insure the conveying of the effluent by pumping from an efficiently designed sewage disposal plant outside of the water-shed of a portion of New York City's water supply.

This idea, which is gaining momentum, carried to its absurd but logical conclusion, means that no city in the United States can discharge its sewage into any body of water which is not tide-water. It is the speaker's opinion that the matter is sufficiently important to demand the serious thought of this Society, and he would suggest that the attention of the Board of Direction be especially directed to this matter, to the end that some intelligent measures may be taken by the Society to counteract this dangerous proposition.

Mr. Burgess. PHILIP BURGESS,* M. AM. SOC. C. E. (by letter).†—The writer has read this paper with much interest. As stated in the paper, it has been the author's purpose principally to discuss the investigations made by the engineers who were retained by the City to determine the best method of obtaining a satisfactory water supply. The writer was retained by Mr. Smith, the contractor for the system which was finally built, principally to conduct the preliminary investigations which Mr. Smith had made before the construction of the plant, and, finally, to prepare the plans from which the system was actually constructed.

* Columbus, Ohio.

† Received by the Secretary, March 21st, 1917.

The following discussion is offered principally for the purpose of describing the preliminary tests and reports made prior to actual construction. Mr. Burgess.

From his position as an outsider, the writer was particularly interested in the opinions of the comparatively large number of engineers obtained before the City authorities finally agreed as to the system which was considered best adapted to serve the needs of the city. The writer was led to believe that perhaps this condition was attributable largely to the fact that the reports of the engineers as submitted were not sufficiently conclusive and convincing, and that they contained serious differences of opinion.

The first report submitted by Messrs. Chapin and Knowles recommended "that the ultimate object to be looked forward to is the establishing of new water-works at the Camden Farm, with the building of either wells or a mechanical filter plant; the former, if investigations show the feasibility, and the latter, if not so found."

Again, on page 16, of that report, it is stated: "It cannot be too strongly urged that, while the well proposition is attractive, further investigations should be made, perhaps in other localities nearer the proposed reservoir site."

It is apparent that, although this report seemed to favor the construction of a well system, it showed that the engineers were not in a position to make a definite recommendation for building such a system without further investigations.

Under date of February 21st, 1910, Messrs. James H. Fuertes and George W. Fuller submitted to the city officials a joint report, in which appears the following statement:

"We find, as will be seen from the estimates of cost of construction and operation, that a first-class well system and a first-class filter system at the Camden Farm are substantially on a parity as to the quality of the water, and as to cost, both of construction and of operation (when a suitable charge for the required land is made against the well system), and that either plant could be depended upon to supply such quantity of water as would be needed by the city for some years. * * *."

The conclusions and recommendations of Messrs. Fuertes and Fuller were very definite, but an examination of their report indicates that these conclusions and recommendations were based on certain assumptions, as shown on page 13 of that report, as follows:

"During wet weather, perhaps when the water in the Ohio River is high, it appears to us that the source of water contained in the pores of the extensive sand and gravel layers is largely river water, but to some extent water from upland sources, the high river stage damming up some of the upland water.

"We are firmly of the opinion that, during low-water stages in the Ohio River, the water which can be drawn from the strata is that

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which has been stored within the pores of the material at times of previous high water in the river. As this stored water is removed, or naturally flows away, we are certain that, during low stages in the river, whatever water enters these sub-surface strata of sand and gravel is largely of river origin.

"Our conclusion, therefore, is that, as a source of supply for a municipality, the water from the underground sources in question, at the end of protracted droughts, must come from the Ohio River through the bottom or sides of the present river bed along the stretch of the bottom lands below Briscoe."

The conclusions of the engineers as to the source of the gravel-water modified their report, as will be shown in the following discussion. On pages 14 and 15, of the report, appear the following statements:

"In the first place, it was found that the water in the porous sand and gravel layers in question did not rise to such high levels as found in the river itself. This is true as indicated by readings in test wells almost at the bank of the river; and shows that, even during fairly high velocities in the river, there is still so much mud and silt deposited on the sides and bottom of the river that a considerable head is required to overcome the friction sufficiently to allow the water, in substantial quantities, to pass through this thin and more or less impervious deposit of mud."

This statement is of interest, in view of the fact that the bottom of the Ohio at Parkersburg is always clean and free from deposits of silt.

In their concluding discussion of the quality of the supplies, the engineers stated as follows:

"There are some slight differences in quality of the water from wells and from filters, the principal feature of which would be hardness on account of the wells yielding water coming largely from the Ohio River at times when it is in flood and fairly soft. We do not attach much importance to this difference because ultimately during low-water periods the well water would also have its source in the Ohio River. So far as quality is concerned, we put waters from filters and from wells in the vicinity of the Camden Farm on a parity. * * *."

In describing the well system and its operation, the engineers recommended the following method of increasing the supply during low-water stages:

"*River Plant.* To insure the removal of mud and silt from the bottom of the river adjacent to the well plant at the latter end of low-water periods, we have considered it prudent to provide for a hydraulic dredge located upon a barge equipped with 25-horse power engines, steam-driven, and which would operate not only the centrifugal pumps for dredging purposes, but also some arrangement for propelling the boat at intervals up and down this distance of 4 000 ft. adjacent to the wells. The suction of these dredges would be specially designed,

resting upon a shoe and with the end attached to a special casting, with a view to removing say for a depth of 1 or 2 in. the material over as wide a strip as possible. It is probable that such a general device could be arranged so as to clean the bed of the river for a width of 20 or 25 ft. and for a length of 4 000 ft. in a working day. This would give an available filtering area of roughly two acres. * * *. Mr. Burgess.

"We estimate that the annual cost of operation and maintenance of a 4 000 000-gal. well water project, exclusive of fixed charges, would be as follows:

| | |
|---|---|
| Fuel, based on gas | $11 680 |
| Pay-roll | 4 000 |
| Supplies, repairs, and renewals (average) | 4 000 |
| River dredging | 3 680 |
| Total | $23 360 |

"We estimate that the annual cost of operating and maintaining a 4 000 000-gal. filter plant in a first-class manner, and of pumping the water to the distributing reservoir, exclusive of fixed charges, would be as follows:

| | |
|---|---|
| Fuel, based on gas | $11 000 |
| Pay-roll | 6 500 |
| Coagulants | 2 500 |
| Supplies, repairs, and renewals (average) | 4 000 |
| Total | $24 000" |

From these citations, it is apparent that the conclusions of the engineers as to the cost of operation and as to the comparative merits of supplies derived from a filter plant or from a well system were largely influenced by their assumption as to the source of the well water. It was this assumption in regard to the source of the well water and in regard to the method of maintaining and increasing the available well water supply during dry-weather conditions, which was not convincing or conclusive to, at least, some of the city officials and members of the Water Commission. If one omits the annual cost of river dredging, estimated at $3 680, from that of operating the well system, it is materially less than that of operating mechanical filters, as determined by the engineers. In other words, omitting such cost of river dredging, and assuming the filtered river supply and well water to be equally satisfactory as to quality, the engineers' conclusions and recommendations for establishing a mechanical filter plant were not warranted.

Messrs. Fuertes and Fuller did not agree with the conclusions of Mr. Clapp, who thought that the ground-water at the wells had its source partly from the sand plain districts and partly from the bottom-lands in the main valley of the Ohio.

As stated by the author, the City finally employed Mr. Samuel M. Gray, who submitted a final report, under date of May 26th, 1910,

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which was so convincing and conclusive, especially as to the merits of the system advocated by Mr. Smith, that the City soon entered into an agreement with Mr. Smith to build his system.

The author has given a very interesting discussion of the preliminary investigations made at Parkersburg to determine the comparative merits of supplies, obtained either by filtering the Ohio River water or by puting in wells adjacent to the river. However, he does not mention any of the preliminary studies made to determine the feasibility of the Smith system. This feature of the work embraced perhaps a more thorough investigation than was made to determine the merits of the well system. Moreover, it is the writer's belief that the final success of the Smith system at Parkersburg is attributable largely to the thoroughness with which the preliminary tests were conducted to determine, first, the feasibility of putting in such a system, and, finally, to determine the best location for the plant.

Briefly stated, the method of conducting the preliminary tests included the sampling of the underflow of the river, as obtained by driving small well points to depths ranging from 3½ to 5 ft. beneath the surface of the bar. Water was pumped from these well points for such a time as was required to obtain a clear sample which was finally analyzed for the purpose of determining its chemical constituents. It was recognized that two important features must be considered in selecting the location for the system, namely, that the underground water must not contain an excessive quantity of iron, and that it must not be excessively hard, as compared with the river water.

The first preliminary tests were made at Parkersburg on August 6th and 7th, 1909. At this time only four tests were made, at stations extending from about 5 000 ft. to 1 000 ft. above the city pumping station. The results of the analyses of the samples collected at this time are shown in Table 1.

The analyses indicated considerable differences in the quality of the water beneath the bar at the different stations. At Station 1, which was near the foot of Neals Island and between the two channels of the river, the ground-water was found to be comparatively hard and of high iron content. This was true also at Station 4, which was 1 000 ft. above the city pumping station. At Station 3, the ground-water obtained from beneath the river was softer than the river water and of low iron content. Near this station, on August 8th, a hole, about 15 ft. in diameter, was dredged in the river bar to a depth of 5 ft. and, in this hole, was placed an 8-in. brass strainer, 8 ft. long, in a horizontal position. The strainer pipe was covered with 18 in. of gravel and about 42 in. of sand. Water was pumped from the strainer by a 2-in. centrifugal pump for the purpose of determining the quality of the water collected in the strainer pipe. Representative analyses of the river water and that obtained from the strainer are shown in Table 2.

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TABLE 1.—RESULTS OF TESTS OF WATER OBTAINED FROM BENEATH THE BOTTOM OF THE OHIO RIVER, BY DRIVING WELL POINTS, AT PARKERSBURG, W. VA.

(Results in parts per million.)

| | | | | | |
|---|---|---|---|---|---|
| Station No. | 1 | 1 | 2 | 2 | 2 |
| Depth of sample below surface of bar | Surface of river | 2.5 ft. | Surface of river | 3.0 ft. | 4.0 ft. |
| Sample No. | 1 | 2 | 3 | 4 | 5 |
| Date of collection | 8/6 | 8/6 | 8/7 | 8/7 | 8/7 |
| Time | 3 P. M. | 4.30 P. M. | 8.30 A. M. | 8.45 A. M. | 9.30 A. M. |
| Temperature, in degrees, centigrade | 28.0 | 23.0 | 26.0 | 23.0 | 23.0 |
| Turbidity | 11 | 5 | 17 | 0 | 0 |
| Free $CO_2$ | 0 | 23 | 0 | 11 | 18 |
| Dissolved oxygen | 9.0 | 2.0 | 7.7 | .... | 1.8 |
| Chlorine | .... | .... | 31 | .... | 14 |
| Iron (Fe) | .... | 5.0 | 0.5 | .... | 6.0 |
| Alkalinity | 15 | 174 | 17 | 57 | 58 |
| Incrustants | .... | .... | 100 | .... | 20 |
| Total hardness | .... | .... | 117 | .... | 78 |

| | | | | | |
|---|---|---|---|---|---|
| Station No. | 3 | 3 | 4 | 5 | |
| Depth of sample below surface of bar | 2.0 ft. | 3.0 ft. | 2.0 ft. | 2.0 ft. | |
| Sample No. | 6 | 7 | 8 | 9 | |
| Date of collection | 8/7 | 8/7 | 8/7 | 8/7 | |
| Time | 11.45 A. M. | 12.30 A. M. | 1.55 P. M. | 3.30 P. M. | |
| Temperature, in degrees, centigrade | 23.0 | 23.0 | 25.0 | .... | |
| Turbidity | 0 | 0 | 0 | 5 | |
| Free $CO_2$ | 30 | 30 | 26 | 9 | |
| Dissolved oxygen | 1.0 | 1.0 | 2.4 | .... | |
| Chlorine | 8 | 7 | 10 | 17 | |
| Iron (Fe) | 0.5 | 0.9 | 1.8 | 1.0 | |
| Alkalinity | 44 | 42 | 75 | 136 | |
| Incrustants | 15 | 5 | 0 | 25 | |
| Total hardness | 59 | 47 | 75 | 161 | |

TABLE 2.—PRELIMINARY TESTS OF SMITH SYSTEM AT PARKERSBURG, W. VA., AUGUST 10TH-15TH, 1909.

(Results in parts per million.)

| | | | | |
|---|---|---|---|---|
| Source of samples | River | Strainer | River | Strainer |
| Sample No. | 10 | 11 | 12 | 13 |
| Date of collection | 8/10 | 8/10 | 8/15 | 8/15 |
| Time | 10.30 A. M. | 10.30 A. M. | 9.30 A. M. | 9.30 A. M. |
| Temperature, in degrees, centigrade | 26.0 | 25.0 | 24.0 | 23.0 |
| Turbidity | 15 | 0 | 1 750 | 0 |
| Free $CO_2$ | .......... | .......... | 2.5 | 23 |
| Dissolved oxygen | .......... | .......... | 7.0 | 3.2 |
| Chlorine | 56 | 24 | 21 | 17 |
| Iron (Fe) | 1.2 | 0.5 | 15.0 | 1.2 |
| Alkalinity | 20 | 24 | 42 | 34 |
| Incrustants | 110 | 85 | 50 | 15 |
| Total hardness | 130 | 109 | 92 | 49 |
| Total solids | .......... | .......... | 749 | 167 |
| Loss on ignition | .......... | .......... | 58 | 28 |
| Nitrogen, as nitrites | .......... | .......... | 0.010 | 0.008 |
| " as nitrates | .......... | .......... | 0 | 0 |
| Oxygen consumed | .......... | .......... | 9.3 | 1.8 |
| Bacteria, per cubic centimeter | .......... | .......... | 8 700 | 82 |
| *Coli* present in | .......... | .......... | 0.05 c.c. | 0.3 c.c. |

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In general, the water obtained from the strainer pipe was entirely clear, even when the river water had a turbidity of nearly 2 000 parts per million; moreover, the total hardness of the strainer water was less than that of the river water. It was undoubtedly true that the success of this preliminary test interested a large number of citizens and officials at Parkersburg, and showed them the possibility of obtaining a satisfactory water supply in this manner.

At the request of Mr. Gray, still further tests were made on May 7th, 1910, to determine the quality of the water in the bar beneath the river and the feasibility of obtaining a satisfactory supply from horizontal strainers placed beneath the bed of the river. At this time, also, by means of a dredgeboat, the material composing the bar was excavated and examined thoroughly to determine its characteristics at numerous points in the river.

As was true during the first preliminary tests, considerable differences were noted in the character of the water obtained from beneath the river at different depths beneath the surface of the bar and at different locations in the bar. The results of the analyses of the samples collected at this time are shown in Table 3.

TABLE 3.—Results of Tests of Water Obtained from Beneath the Ohio River, by Driving Well Points, at Parkersburg, W. Va., May 7th, 1910.

(Results in parts per million.)

| | | | | | | |
|---|---|---|---|---|---|---|
| Station | 1 | 1 | 1 | 1 | 2 | 2 |
| Depth of sample below surface of bar | Surface of river | 2.0 ft. | 3.5 ft. | 5.0 ft. | Surface of river | 4 ft. |
| Sample No. | 1 | 2 | 3 | 4 | 5 | 6 |
| Time of collection | 11 A. M. | 11 A. M. | 12.20 P. M. | 1 P. M. | 4 P. M. | 4:30 P. M |
| Temperature, in degrees, centigrade | 15.5 | 14.0 | 13.3 | 13.3 | .......... | 13.3 |
| Turbidity | 35 | 50 ± | 10 ± | 5 ± | 40 | 0 ± |
| Free $CO_2$ | 1 | 4 | 4 | 5 | .......... | 3 |
| Dissolved oxygen | 9.1 | .......... | 2.6 | 0.7 | .......... | 2.2 |
| Chlorine | 15 | 9 | 9 | 10 | 13 | 13 |
| Iron (total Fe) | 4.0 | 2.0 | 0.2 | 0.1 | 1.5 | 0.3 |
| Alkalinity | 24 | 128 | 142 | 223 | 20 | 40 |
| Incrustants | 23 | 20 | 15 | 10 | 27 | 12 |
| Total hardness | 47 | 148 | 157 | 233 | 47 | 52 |
| Total solids | 176 | 226 | 203 | 289 | 136 | 169 |
| Loss on ignition | 68 | 42 | 43 | 17 | 29 | 46 |

After the City had entered into a contract with Mr. Smith to build his system, still further tests were made by the writer in June and July, 1910, for the purpose of determining the best location. More than seventy-five well tests were made at intervals of about 100 ft. over the river bottom, and a small pile-driver was rigged up on a small flatboat to assist in driving the well points. The sampling stations were located by stadia measurements from a base line on the bank of the river. By this method of testing, it was learned that the best location for the system proposed was not opposite the Camden

Farm, but several thousand feet below this farm at a point where the material composing the river bar was very satisfactory and the underlying water was of low iron content and softer than the river water. Consequently, this location was selected as best adapted for the system. Mr. Burgess.

Further studies of the design of the proposed strainer system indicated the desirability of dividing it into a number of units with independent suction lines, rather than to make only two units, as was contemplated in the original plan. Moreover, on account of the large losses of head which would develop during back-flushing, it was realized that the pumping station should be at the nearest possible point to the strainer system. As shown by the author, the strainer system, as constructed, was composed of five individual units, each connected by separate 18-in. suction lines to the main suction of the pumps, and the latter were near the river bank opposite the center of the strainer system.

It is the writer's belief that two important features of construction have contributed especially to the success of the strainer system, namely, the provision of a deep bed of clean, coarse gravel over the entire area covered by the strainer system, and the reduction in the area of the strainer pipes at their connections to the main distributing pipes to a diameter of 2½ in. The deep gravel layers have increased the capacity of the strainer system materially, and also the reduction in the area of the connections of the strainer pipes has also increased the efficiency of back-flushing the system, by introducing a loss of head at this point, thus causing a better distribution of the wash-water.

In view of the possible differences of opinion as to the source, or sources, of the water in the adjacent gravel bed along the Ohio River, and also in the gravel beneath the river, it may be noted, first, that the entire river bottom at Parkersburg is covered to a depth of approximately 18 in. with very hard impervious gravel layers firmly cemented together. In fact, in some places, during the preliminary testing, the gravel layers were so tight that it was found impossible to pump water from the well points after they were driven. This feature is of interest in considering the recommendations contained in one of the preliminary reports to use a hydraulic dredge of such design that it would remove material from the river bar to a depth of 1 or 2 in. Experience in dredging material from the bottom of the Ohio shows that it is impossible to use hydraulic dredges in this river above Portsmouth, Ohio, on account of the compactness of the surface material.

Moreover, some light as to the probable source, or sources, of water obtained at shallow depths from beneath the river may appear

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from Table 4, showing results of analyses of samples of water collected by the writer at Sistersville, W. Va., on August 1st, 1910.

TABLE 4.—RESULT OF ANALYSES OF SAMPLES OF WATER FROM THE OHIO RIVER AT SISTERSVILLE, W. VA., AUGUST 1ST, 1910.

(Results in parts per million.)

| Stations | 1 | 2 | 3 | Ohio River |
|---|---|---|---|---|
| Depth of water, in feet | 4.7 | 3.9 | 3.3 | .... |
| Temperature, in degrees, centigrade | 23.5 | 23.5 | 23.0 | 26.0 |
| Dissolved oxygen | 0 | 0 | 0 | .... |
| Total solids | 7 300 | 6 400 | 6 100 | 262 |
| Iron (as Fe) | 1 100 | 1 300 | 900 | 0.7 |
| Incrustants | 475 | 482 | 475 | 62 |
| Alkalinity* | 147 | 150 | 156 | 36 |
| Total hardness | 622 | 632 | 631 | 98 |
| Chlorine | 4 500 | 4 250 | 4 500 | 27 |

* Alkalinity, August 3d, 1910, Sample No. 1 = 28; No. 2 = 5; No. 3 = 44.

NOTE.—Samples of ground-water were obtained 4.0 ft. beneath the surface of the bar.

The samples in Table 4 were obtained from the bar at different stations near the center of the river by exactly the same method as that used at Parkersburg, and it is of interest to note that, even at shallow depths beneath the surface of the bar, the ground-water was of remarkably high mineral content. The samples were clear when collected, but in 15 min. they looked like tomato ketchup.

The writer has made a large number of other investigations, along similar lines, to determine the character of the water obtained at shallow depths beneath river bars, principally at Owensboro, Ky., Gallipolis, Ironton, Zanesville, Portsmouth, and Newark, Ohio, and Wheeling, W. Va., and, in all cases, the results obtained have been generally similar to those at Parkersburg. The tests have indicated great variations in the quality of the water at different locations and at different depths of sampling. The writer's conclusion is that the underground water, even at shallow depths, is always of very different quality from the river water, especially during low stages of the streams; also, that during low stages it cannot have its source in the river, but must come from adjacent higher territory and flow toward the river.

An interesting feature was developed at Parkersburg where, in some cases, the water obtained from the well points was of very high iron content and contained only 5 parts of incrustants when analyzed by the usual methods. Its alkalinity was 55 parts. Two or three such points were included within the area covered by the strainer system, and excavations disclosed the fact that a log or trunk of an old tree had been deposited and covered within the river bar. Immediately around these logs, for distances of about 10 ft., the surrounding gravel and sand was filled with iron. When the log and iron deposits were removed, the quality of the water obtained from

the ground changed, and resembled that flowing from adjacent areas. Its iron content was reduced to about 0.3 part per million, the incrustants increased to about 70 parts, and the alkalinity was reduced to about 40 parts. The writer has no explanation for this phenomenon. Mr. Burgess.

In that part of the report of Messrs. Fuertes and Fuller which discusses the quality of the ground-water obtained from the test wells it is stated that, in some instances, the ground-water contained objectionable quantities of iron "due to the water dissolving iron during the long intervals of standing in contact with the metal;" and, again, that the large quantity of iron in the well water at the steel plant is explained "partly by the flow of water through a long line of pipe." The writer believes that these statements should not go unquestioned, because undoubtedly the high iron content of the ground-water was not attributable to its contact with iron pipes, but the iron was present in the water in the gravel strata. It is significant that the test wells producing water of high iron content were opposite those points in the river where the well point tests indicated that the underground water beneath the river, also, was of high iron content. It is a common popular fallacy to attribute the iron found in some well waters to their contact with the pipes.

Moreover, the writer would question the judgment which dictated placing the wells along the river bank 400 ft. apart. Such a spacing would seem to be too great, and would not make available all the water contained in the gravel strata. Although the water levels in the test wells indicated that the contours were parallel to the river bank, it should be noted that these levels were taken during comparatively high stages of the river. It is the writer's opinion that if levels had been taken during low stages, it would have been found that the ground-water table had a different slope, and that the flow of water would have been slightly down stream toward the river bank. This would conform more nearly to the conclusions of Mr. Clapp, as stated in his report.

It is believed that experience at Parkersburg warrants the conclusion that each of the following features is of importance and should be considered in the construction of strainer systems or infiltration plants of this type:

1.—The presence of a permanent bar of sand and gravel of suitable size within a reasonable distance of the pumping station to which the water is to be carried;

2.—The constant submergence of the bar from which the supply is to be obtained;

3.—The velocity of flow in the stream, which velocity must be sufficient to maintain the bar clean and free from silt and other drift;

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4.—The depth at which the collecting units are placed with relation to the bottom of the stream; and

5.—The character of the water which is found beneath the bottom of the stream.

Experience at Parkersburg has shown, also, that it is very important that there should be a suitable meter or measuring device on each suction line from each section of the system. Without such meters, it is impossible to operate the different sections of the system at the same rate. Immediately after back-flushing a section, the loss of head is very much decreased, and it is difficult to avoid excessive rates of filtration from the newly washed unit without a suitable meter.

Although this paper was presented primarily with a view of discussing the preliminary investigations by engineers to determine the merits of the various water supplies available at Parkersburg, it may be well to mention briefly the merits of the system actually built. Its obvious advantages are its low cost of operation and its simplicity and ease of operation. The annual cost of operation is only one-half of the estimated cost of a suitable well system or rapid sand filter plant, as discussed in the preliminary reports of the consulting engineers. Moreover, the operation of the plant is accomplished successfully without expert control.

Though it may be true that the results accomplished by the plant are not within certain arbitrary standards adopted by Federal authorities, it is to be noted that the plant was constructed under more severe guaranties of efficiency than have ever been enforced at any other water purification plant in the United States. Moreover, the plant was accepted on the basis of these guaranties and on the basis of thorough tests of operation by a competent representative of the City.

It is the writer's belief that the City should erect a suitable chlorinating plant as an additional precaution in securing always an entirely satisfactory supply. In doing so, however, it should be recognized that the city is taking no additional precautions other than those now adopted by very nearly all cities which obtain their water supplies from either rapid or slow sand filters. There is no doubt that the results obtained by the present plant are comparable with those from rapid sand filters, and that the cost of operation is so low that it makes the type now in use at Parkersburg of very great advantage to any city where conditions are such as to permit of its satisfactory installation.

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JAMES H. FUERTES,* M. AM. SOC. C. E.—Messrs. Fuller, Hill and Potter, having discussed quite fully the principal and more important matters dealt with by Mr. Hall, a few remarks concerning experiences with somewhat similar methods of securing water from river beds may be of interest.

* New York City.

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A number of water companies and cities on the Allegheny and Ohio Rivers in the vicinity of Pittsburgh take water from these rivers by a plan having general features in common with those of the plan adopted at Parkersburg.

About 10 years ago three citizens of Wilkinsburg, Pa., a suburb of Pittsburgh, brought suit against the Pennsylvania Water Company to compel that company to furnish pure water, as required by the terms of its charter. The suit was rather unique, and was one of the few recorded in which a decree has been entered directing a company to secure and provide a sufficient supply of pure and wholesome water. The supply against which complaint was made was drawn from gravel-covered cribs in the Allegheny River bottom. The first supply crib, built in 1897, was 32 ft. wide, 308 ft. long, and 5 ft. high, made of 2 by 6-in. and 2 by 8-in. planks, placed edgewise and flatwise, and separated by intervals of 2 in. A 24-in. pipe extended from the interior of the crib to the pump-well on the shore. The crib was sunk in a dredged excavation, about 300 ft. from the north shore of the river, and at a depth placing the top of the crib about 5 ft. below the natural bed of the river. The excavation was refilled and the crib covered to a depth of 1 ft., with 2-in. gravel, and then with from 4 to $4\frac{1}{2}$ ft. of sand and fine gravel, such as had passed through a $1\frac{1}{2}$-in. mesh screen. The depth of the water over the crib was ordinarily from 8 to 10 ft.

It was shown by analysis of the water that the supply was drawn partly from the river by filtration through the gravel over the cribs, and partly from the sub-stratum of ground-water in the 30-ft. bed of gravel and sand in the river bottom.

In 1902, two additional cribs, each 48 ft. wide, 408 ft. long, and 5 ft. deep, were put in service. They were constructed in the same general manner as the first, but excluded from the gravel covering stones larger than 1 in. in diameter. Three cribs were sunk side by side and 10 ft. apart, and a line of 42-in. pipe, decreasing to 24-in. in diameter, was laid in the 10-ft. space between the cribs, with four short pipe connections therefrom to each crib about equally spaced throughout their length. The main 42-in. pipe was extended under the river bottom to the pump-well on shore.

The water from the cribs discharged under its own head into the pump-well, and was pumped thence to a reservoir, from which the street mains distributed it to the consumers.

After periods of operation of varying lengths, the surfaces of the gravel over the sunken cribs would become more or less choked with silt, dirt, and other matters suspended in the river water, and gradually fail to yield a sufficient supply. To relieve this shortage, the company had recourse to cleaning or washing the filter areas with water. The filtering area immediately over the cribs—with all three

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cribs in use—was about 1.13 acres; undoubtedly, however, water entered the cribs from the bottom and from areas outside of the boundary lines of the cribs. It is not possible, therefore, to state the rate of filtration per unit of area per day, except that it was less than about 7 000 000 gal. per acre per day by the quantity derived from sources other than the filtering area over the cribs.

The plan followed in cleaning these filter areas was to drag back and forth across the area a rake having a 2-in. pipe header, 3 ft. long with seven teeth, 15 in. long, each perforated with two $\frac{3}{16}$-in. holes on the front and one in the end, water being forced down through this rake at a pressure of 125 lb. per sq. in. by a pump on a barge. The rake was dragged across this area, the wash-water rising up through the sand and bringing the mud with it, the principle resembling that by which the Blaisdell filter washer is operated. The river current was depended on to carry the washings down stream and away from the washed areas. The rake was dragged back and forth across each area three times, by hand windlasses on barges anchored on each side of the areas being washed, before a new area was attacked.

Raking, or scouring, was practically continuous from May to November, 1905, and from May to December, 1906, except as interfered with by uncontrollable conditions, such as river floods. In 1906, the surface was examined several times by a diver, who reported considerable disturbances of the sand surface, and, under his guidance, depressions were filled up with gravel and sand deposited from a barge through a tube. Frequently, the diver rode on the rake, as it was dragged back and forth, to press the teeth down into the sand. It was stated that the teeth usually penetrated the top of the gravel bed to a depth of from 8 to 12 in. While the process of raking or scouring was going on, the cribs were kept in constant use. The disturbance of the surface and the destruction of the silt formation thereon permitted a largely increased volume of water to pass freely through to the cribs, through the washed areas, and during and immediately after these periods of washing, the water was easily shown to be very little different, in character, from the raw river water.

Much evidence was introduced at the trial as to the quality of the water, both as to its chemical and biological features, and it was shown that there were times when, for a number of consecutive days, the quality of the water would be excellent; all of a sudden, however, some change in conditions of river flow would cause the silt to be washed from the river-bed over the cribs, or parts of their areas, leaving places where the river water could penetrate to the interior of the cribs in a practically unchanged and polluted condition.

In 1905 and 1906, typhoid fever in Wilkinsburg became epidemic, and just prior to the bringing of the suit it had the highest typhoid fever case and death rate of any town in the United States. Very

heavy rates also prevailed in the 37th Ward of Pittsburgh, and in Swissvale Borough, also supplied with water by the same company. As a result of this alarming condition, protests were lodged, and, finally, recourse was had to the Courts to compel the Water Company to safeguard adequately the quality of the water. The case, recorded as E. Z. Peffer *et al. vs.* Pennsylvania Water Company, was No. 396, tried in the August term, 1906, of the Court of Common Pleas, No. 3, of Allegheny County, Pennsylvania, Miller, J., specially presiding. The conclusions of law, in this case, as drawn by Judge Miller, were: **Mr. Fuertes.**

*First.*—The fact having been found that the water complained of, as furnished by the defendant, is not reasonably pure and wholesome, it follows that the plaintiff's bill must be sustained.

*Second.*—An interlocutory decree will be entered, directing it to secure and provide a sufficient supply of pure and wholesome water; further directing that it shall, within three months from the date of decree, file a statement of the steps it has taken, and purposes to take, in compliance with the requirements to furnish reasonably pure and wholesome water, upon the submission of which the plaintiff may file a reply or answer, as he may deem advisable. It will be further decreed that this case be retained for such further proceedings as may be necessary to insure its performance, and to enable the Court to exercise the jurisdiction conferred by this Act of Assembly under which this action is brought, with liberty on the part of either of the parties to apply to Court for further orders and decrees as may be necessary and just.

*Third.*—The defendant to pay the costs.

The meagerness of the data regarding the details of design, construction, and operation of the Parkersburg plant, in Mr. Hall's paper, makes extended discussion of it impossible. Those elements of design which would give control over the process of filtration are scarcely mentioned. Enough is stated, however, as to the diameters of the main pipes leading from the strainer system, and as to the sizes of the laterals and their perforations to indicate:

1.—That hydraulic conditions must exist, which would be avoided in a properly designed filter plant.

2.—That there can be almost no control over the effectiveness of back-flushing, or washing of the filter areas.

3.—That the loss of head must vary between quite wide limits at various parts of these areas during filtration, and after partial clogging shall have taken place, and that, therefore, parts of the areas must be passing the water at much higher rates than others, and, consequently, must be doing less effective work than planned.

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4.—It is also evident that the back-flushing is likely to blow up and overwash parts of the areas, and leave other parts badly clogged.

5.—That floods in the river may scour, and probably have frequently scoured, holes in the filters—possibly almost down to the strainers.

6.—That the whole filter area is inaccessible, and utterly beyond inspection and repair during floods, the times when the water is worst and the need of protection greatest.

The Parkersburg plant may be expected to give fair water when the river is in its best condition, and, at such times, it is almost good enough to use without filtration. When the river water is turbid and dangerously polluted, the water delivered by the plant may or may not be safe. Whether or not it is safe will be beyond the knowledge of the men operating the works; and, if it is unsafe, it is entirely beyond their power to make it so by any agency or device inherent to or part of the plant itself or of its design.

Judging by the discoveries of Mr. Leland, when making the inspections in September, 1916, referred to by Mr. Hall, the Parkersburg plant is experiencing difficulties similar to those which led ultimately to the virtual condemnation of the Wilkinsburg works and to the erection of a properly built mechanical filter plant in its stead.

The cost of construction and operation of the works, as built at Parkersburg, are given by Mr. Hall as $284 171.22 and $11 892.46, respectively. The annual cost was based on the pumpage of an average of 3 000 000 gal. daily. In the Fuertes-Fuller report, the costs were given as $165 945 and $24 000, respectively, the latter figure being the estimated annual cost of operation for a daily yield of 4 000 000 gal. Reduced to the basis of 3 000 000 gal. daily, for comparison with the figures given by Mr. Hall for the Parkersburg plant, and adding to the cost of construction certain items included by Mr. Hall, and with interest at 7% in each case, the annual operating and maintenance costs of the two plants would be practically equal. The Smith system, therefore, has required at Parkersburg a greater investment, saves nothing in annual expense, and gives to the city far less security against water-carried diseases than would have resulted from the establishment of a properly built and operated mechanical filter plant, such as recommended and described in the Fuertes-Fuller report.

Mr. Hall gives, among the reasons advanced by Mr. Gray for the adoption of the Smith system for Parkersburg, that,

"If properly built and operated, this system will furnish water of a better quality than a mechanical filter plant, as regards steam-raising purposes, on account of the slight increase in permanent hardness which is caused by the use of a coagulant."

Although it is true that the use of aluminum sulphate as a coagulant will result in the conversion of a portion of the alkalinity into the sulphate or incrustant form of hardness, an examination of the analyses in Appendix I shows that throughout January, 1914, the total hardness of the water from the Smith system was about 87 parts per million, as against about 74 parts per million for the river water, this increase being due to the increased alkalinity of the sub-river ground-water. The increased quantity of soft scale due to the extra 13 parts per million of hardness of this ground-water is probably fully as disadvantageous from a steam-raising point of view as would be the conversion of about 5 or 6 parts per million of the low alkalinity of the river water into a proportional quantity of incrustants by the application of aluminum sulphate used in connection with mechanical filters. For summer conditions, when both the river water and the water secured by the Smith system would be much harder than reported in the January analyses, the advantages would probably lie strongly with the mechanically filtered water. Mr. Fuertes.

And, further, with the mechanical filters, the operatives would have access to and control over the filters and their efficiency at all times. With the Smith system, the whole delicate process of filtration is beyond reach, often buried under floods and subject to disturbance by wash and scour, or to irregular deposition of mud on the filter area, with consequent imperfect purification, due to varying and excessive rates of filtration on parts of the area smaller than the total required.

Taking everything into account, the speaker is not convinced, by the data brought forward, that the proper decision was made at Parkersburg when the Smith system was adopted in preference to a properly designed and constructed mechanical filter plant.

It is much to be regretted that so little information has been furnished as to the operation of and results obtained by the Parkersburg water-works plant. Fancied security has led to neglect of vigilance, as represented by systematic, continuous, analytical studies of the river and secured waters, and more frequent examinations of this nature, as recommended by Mr. Hall, should certainly be made by the authorities.

The ground-water studies, made at Parkersburg, and referred to by Mr. Hall, show a number of interesting facts. Figs. 3 to 12 were selected from among 32 such diagrams in the Fuertes-Fuller report, and seem to have been well chosen to exhibit the interesting phenomena of filling and emptying the subterranean storage spaces in the voids in the gravel underlying the flat lands along the river north of Parkersburg.

It will be noticed that the water finds its way into the ground, from the river, with comparative difficulty, as judged by the steep

Mr. Fuertes. slope of the ground-water in the neighborhood of the river banks, but that it makes its escape back into the river, when the latter falls, with comparative freedom. This is as would be expected, the surface of the river bottom and banks having become clogged with mud (strained out of the river water as it filtered out through the banks and bottom) on a rising river, but being cleansed, or washed free of mud to a greater or lesser extent, on a falling river, by the passage of the ground-water back again into the river.

The contours on Figs. 16, 17, and 18 show the gradual change in direction of the flow of the ground-water away from and toward the river above Parkersburg. Fig. 12, which was reproduced from among the diagrams in the Fuertes-Fuller report, occupies a position between Figs. 17 and 18. An examination of these diagrams shows the following interesting conditions.

On January 13th (Fig. 16) the river, after having reached its lowest point from a previous rise, was just beginning another rise which, by the 15th, had reached 9 ft. Observing the ground-water contours on this diagram, it will be seen that the general trend of the ground-water was down stream and toward the Ohio River, the 571-ft. contour being the farthest down stream and the 574-ft. one the farthest up stream, all these contours lying in approximately parallel lines, and indicating a motion diagonally down stream toward the river.

On January 17th, the river began another rise, and, within the next few days, reached an elevation of 597.5 ft. On January 19th (Fig. 17) it stood at Elevation 592. It will be observed that the contours of the surface of the ground-water had changed entirely, swinging around parallel to the shore with the highest contours (Elevation 579) along the river bank, and the lowest still at Elevation 572, on a line parallel with and about 2 000 ft. distant from the river bank. By the 24th, the river surface had fallen to 692.5 and the ground-water near the river bank had risen to 584, or 14 ft. higher than on January 13th, whereas, along a line practically parallel with, and some 2 500 ft. east of, the river bank, the ground-water level had risen only about 1 ft.

On February 5th (Fig. 18) the river still continued to fall, the ground-water which had been piled up to Elevation 584 on January 24th had fallen at the river bank to an elevation of only 573, gradually rising to an elevation of 575, about 2 000 ft. back from the river and dropping to 574 a few hundred feet further east. In this case the water was slowly flowing in both directions from a line parallel to and about 2 000 ft. back from the river bank.

It is very fortunate that this flood occurred during the time that the tests were being carried on at Parkersburg, as the occasions on which such phenomena can be observed are relatively rare.

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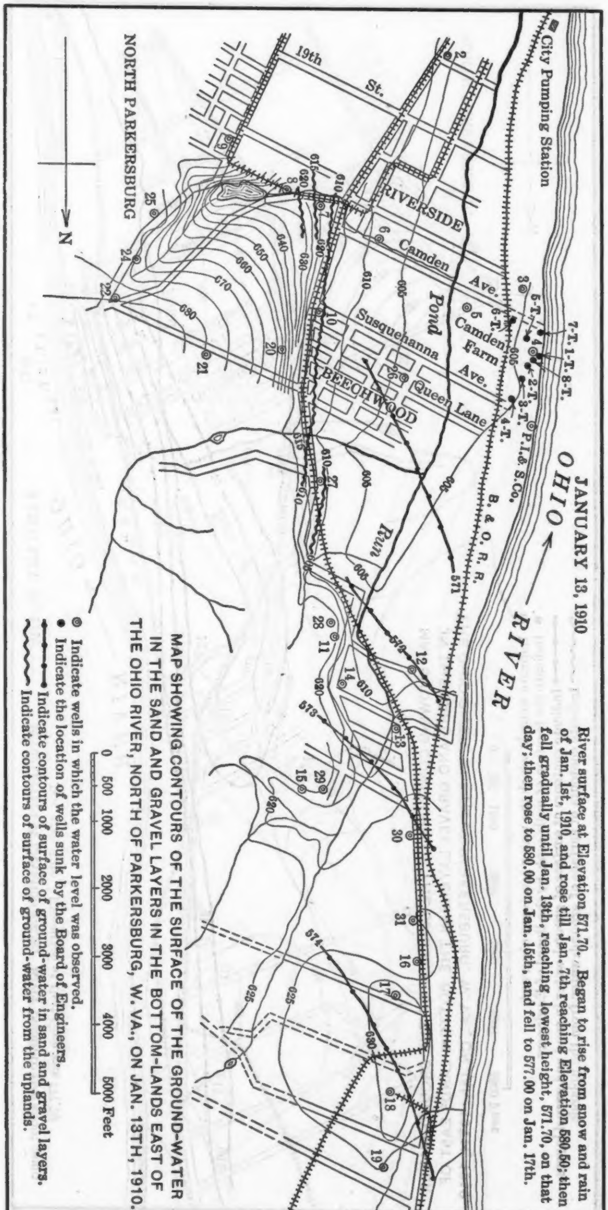


FIG. 16.

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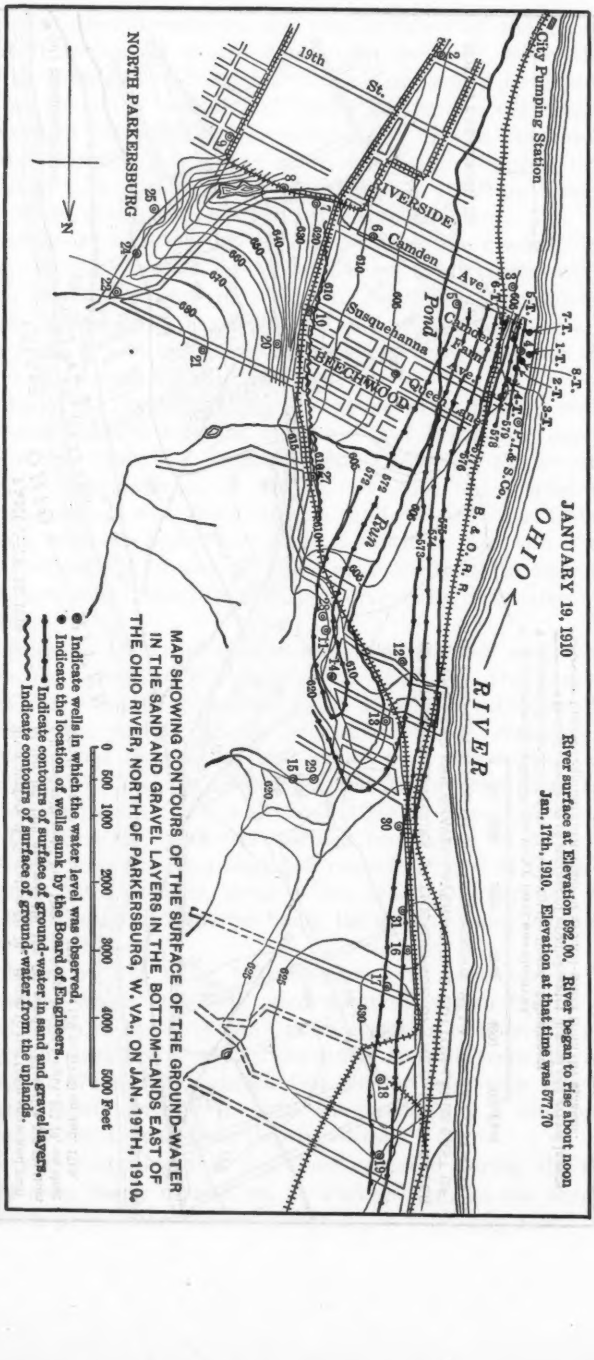


FIG. 17.

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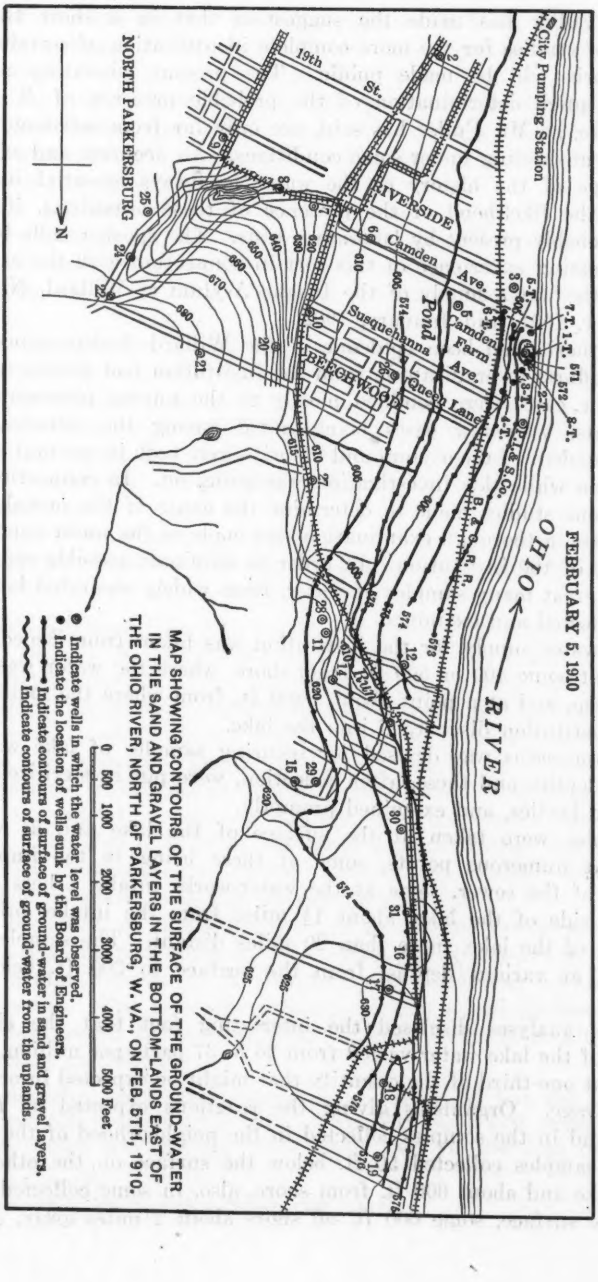


MAP SHOWING CONTOURS OF THE SURFACE OF THE GROUND-WATER IN THE SAND AND GRAVEL LAYERS IN THE BOTTOM LANDS EAST OF THE OHIO RIVER, NORTH OF PARKERSBURG, W. VA., ON FEB. 5TH, 1910.

0 500 1000 2000 3000 4000 5000 Feet

Indicate wells in which the water level was observed.
Indicate the location of wells sunk by the Board of Engineers.
Indicate contours of surface of ground-water in sand and gravel layers.
Indicate contours of surface of ground-water from the uplands.

FIG. 18.

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Mr. Fuller has made the suggestion that in a short time the results of studies for the more complete identification of certain forms of bacteria will be made public. The present laboratory methods for the quick determination of the probable presence of *B. coli* in water, are, as Mr. Fuller has said, not only far from satisfactory, but actually misleading under some conditions. An accurate and extensive knowledge of the history of the water is always essential in determining the likelihood of the presence of these organisms, if shown to be probably present by laboratory tests. The speaker calls to mind an interesting experience of this sort in connection with the examination of the water supply of the Insane Asylum at Willard, N. Y., in December, 1901, and January, 1902.

Diphtheria had been epidemic at the Willard Asylum since 1897, and all efforts to eradicate it from the institution had proven unavailing. The cases were confined mostly to the nurses, physicians, and attendants, very few cases being noted among the inmates. The superintendent of the asylum and his wife were both in quarantine with diphtheria while this investigation was going on. In connection with the various studies made to determine the cause of the prevalence of the disease, a thorough examination was made of the water supply and sewerage of the institution. In order to eliminate possible sources of error, a great many samples of water, from widely separated locations, were collected and examined.

The water supply for the institution was taken from Seneca Lake at a point some 400 or 500 ft. from shore, where the water was about 30 ft. deep, and at a point about 1 800 ft. from where the main sewer of the institution discharged into the lake.

An apparatus was devised for securing samples of lake water at various depths, and these, after collection, were put in sterilized, glass stoppered bottles, and examined promptly.

Samples were taken at the surface of the lake and at various depths at numerous points, some of these being in the immediate vicinity of the sewer, some at the water-works intake, others on the opposite side of the lake, about 1½ miles from the intake, others at the ends of the lake, more than 20 miles distant. The samples were collected at various depths, from the surface to 300 ft. below the surface.

These analyses disclosed the interesting fact that the chlorine content of the lake water varied from 36 to 37 parts per million, which was about one-third of the quantity that might be expected in ordinary town sewage. Organisms giving the reactions expected of *B. coli* were found in the samples collected in the neighborhood of the sewer, also, in samples collected 50 ft. below the surface on the other side of the lake and about 600 ft. from shore, also, in some collected 50 ft. below the surface, some 600 ft. off shore about 2 miles away, and in

a number of other places. Previous analyses made in 1899 had disclosed the suspected presence of *B. coli communis* in samples 1 300 ft. out from shore, on the line of the hospital's sewer, at a depth of 20 ft.; ½ mile from shore at a depth of 150 ft.; ½ mile from shore on a line with the water-works intake at a depth of 400 ft.; 500 ft. deep off shore about 5 miles south of the Asylum, and in the center of the lake at a depth of 400 ft. Mr. Fuertes.

In the analyses made in 1901-02, it was pretty well established that the organisms giving the reactions for the differentiation of the *B. coli communis* were probably washed into the lake from the cultivated fields along the sides, and bore a close relation to the rust on grain.

The high chlorine in the lake water was caused by the presence of underground salt beds in that vicinity; the normal chlorine for that region would not be more than 7 parts per million.

Without knowing all the conditions, the analyses of Seneca Lake water might easily have been mistaken for a highly sewage-polluted water; a conclusion which the most ordinary common sense would refuse to accept.

More positive methods for identification of sewage bacteria in waters have long been needed, and water-works engineers will welcome the greater certainty promised by Mr. Fuller's predictions.

Morris Knowles,* M. Am. Soc. C. E., and J. D. Stevenson,* Assoc. M. Am. Soc. C. E. (by letter).†—This paper is of especial interest to the writers, because of the part they took, in 1908-09, as Consulting and Resident Engineers, respectively, of the Water-Works Commission, in the early studies of the water supply at Parkersburg and the proposed new systems. They believe that a statement of some of the considerations which led to the recommendation to develop a well supply, instead of the system described by the author, will be of interest. The author is to be commended for collating and bringing before the Profession the various studies and reports which have been made in reference to this interesting situation. Messrs. Knowles and Stevenson.

No discussion of this subject would be complete without mention of the lively interest taken in these early investigations by the late United States Senator J. N. Camden, and continued, at his death, by his estate. The generous attitude of this public-spirited citizen can be best stated by quoting from an open letter from him to the city officials, as follows:

"I, therefore, respectfully propose, in order to test the question of natural filtration, that I will, at my own cost and expense, sink and test wells, both on the river front and on the foot of Neal's

* Pittsburgh, Pa.

† Received by the Secretary, March 21st, 1917.

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Island, for which I already have the permission of the owner, to ascertain the quality and quantity of water that can be obtained by natural filtration, and to commence as soon as the engineers selected by the city authorities and water-works commission are ready to direct and superintend the tests to be made. This will cost the city nothing and will no doubt be of value in arriving at conclusions. I will also add that, should the city desire to locate its plant or pumping station or wells upon any ground owned by me, I will donate to the City all the ground it may need for water-works purposes."

In pursuance of the engagement of the writers as engineers, a report was made on August 24th, 1908, covering an investigation of ten possible developments and including a recommendation that either a mechanical filter plant or a system of wells be adopted as a source of supply. A definite choice between these two projects was not recommended at that time for two reasons: First, because the original sum of money appropriated to the Commission did not permit of drilling and testing wells; and, second, because after the money donated by Senator Camden for this purpose was made available, it was concluded "that the time was not propitious for either drilling or testing wells, as both the river and the ground-water level were too high to justify any conclusion from a test as to safe yield."

The investigation of the wells was only postponed, however, being carried out in the succeeding warm, dry season, at a time when both surface and ground-water levels were low. The conditions at the time of the tests were most favorable to a safe conclusion, as the river was lower than it had been for many seasons. The results of these tests warranted the recommendation of a water supply to be drawn from wells on the Camden Farm, which was reported on January 25th, 1909.

The topography, geology, and history of the Ohio Valley give evidence that, at one time, either the Little Kanawha River or Worthington Creek, or both, flowed through a gap in the hills and into the Ohio River at a point near the present settlement of Beechwood. In later years, either the Little Kanawha River straightened its course and shifted to its present location, while Worthington Creek occupied in reverse flow what had been the Little Kanawha River bed; or else, Worthington Creek cut a new course into the Little Kanawha River and thence into the Ohio. In any case, the gap in the hills became filled with sand and gravel, forming what are now known locally as the "Sand Plains", separated from Parkersburg by the rocky promontory known as Boremans Hill.

Evidence is conclusive that in times past the Ohio skirted the West Virginia shore and, at one time, occupied the area now forming the east shore terraces, on which are the Camden Farm and the settlements of Beechwood, Vienna, and Brisco. It is also evident by the

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slope of the rock that, in its former location, the Ohio River was at a lower elevation than at present, the intervening space between the present and the former beds having been filled with a water-bearing sand and gravel formation. The trough made by the intersection of the rock beds sloping from the two shores is under the east shore terraces, and, at the Camden Farm and for many miles up stream, probably has the same general course as Pond Run.

The flow of underground water normally follows the general inclination of the rock, and is outward from the surrounding hills. When the flows from two hills meet, the resultant flow is upward, and if it is abundant and the head is sufficient, an Artesian flow will result. Such is the origin of much of the flow in Pond Run. An inspection of the bottom and sides of the Run and feeders will disclose many springs, and these have been found from above Neal's Island to a point below the Camden Farm.

It would not be expected that there would be a large flow parallel to the river in the sand and gravels directly over the rock, but rather that something like an underground reservoir would be formed, overflowing into the river and to the surface of the terrace. However, in this particular case, the gravel beds are topped by impervious clay and loam. As a result, the head due to the higher level of the ground-water on the "Sand Plains" causes flow in the direction of the inclining floor of the gravel beds, which is not only toward the river, but also down the valley. This was the basis for the proposed development by two lines of wells, one parallel to, and another at right angles to, the river, thus intercepting whatever water flowed toward or parallel with the river.

It will be noted that the top of the gravel beds at the river's edge is very little higher than the low-water level in the river. The observations made by the writers indicate that the underground reservoir is not fed to any great extent by infiltration from the river, but probably receives most of its water from rainfall on the terraces, on the "Sand Plains", and from the 26 sq. miles of drainage area tributary to Worthington Creek, and probably some from entrance at the rock riffles in the Ohio some distance up stream.

In Figs. 3 to 10, inclusive, it will be noted that the surface of the ground-water as shown is always above the line of demarcation between the water-bearing sand and gravel and the impervious loam and clay; and that, in no instance, did the pumping lower the surface of the ground-water into the sand and gravel. It may be concluded, therefore, that, even at high stages of the river, no large quantity of river water would be forced into the sand and gravel. For these strata would always be filled, and, as the impervious clay and loam above would not absorb water in quantities rapidly, the rise in the river

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would only increase the head on the water in the sand and gravel, raising the level of the water in the observation wells correspondingly.

Referring to Fig. 11 the following explanation appears in the writer's report:

"When pumping was checked in amount or stopped entirely, the water quickly rose in the well being pumped, indicating a lack of strainer openings for the free access of the water."

In the absence of long-continued tests, the quantity of water which may be procured from any formation is largely a matter of speculation. The planning of a water supply from wells in any undeveloped territory, therefore, should provide, where possible, for permitting easy future adaptation to other sources of supply, in case this should become necessary. The conditions on the Camden Farm were particularly advantageous for such a water-works development; and, although, as a result of the tests and the comparison of conditions with those in somewhat similar deposits elsewhere, the conclusion reached by the writers, that wells, properly located, drilled, and developed, would yield continuously, for a sufficient period, all the water to be reasonably demanded by the City of Parkersburg, at the same time, the various units could be designed so as to be readily adapted, with a minimum monetary loss, to use in connection with a filter plant, to provide against the remote possibility of failure of the wells.

The estimated cost of the construction recommended by the writers was $220 865. Of this sum, 54% was for distributing reservoir, reinforcing mains, etc., and would be equally useful in connection with the water-works then existing, or with any other that might be designed; 33% was for pumping stations and equipment—structures equally useful, either with a filtration plant, or with a well supply; and only 13% was allotted to equipment useful for a well supply alone. Thus, it is evident that, even if the wells had been abandoned at some future time, the loss on the investment would have been trivial, and fully met in such times by the lesser cost of operating the well system.

Among the various developments studied in connection with the writers' investigation at Parkersburg was that known as the Smith infiltration system. The four principal reasons advanced at that time by the promoters for its adoption were: Economy in installation and operation; purified river water and not hard ground-water; ease of repairs and replacements; and reliability of supply. Instead of confirming any one of these points, the investigation of the writers decidedly discredited all of them and, further, led to the conclusions: that the infiltration system, being submerged in the bed of the river, is subjected to the shifting of the river bottom, and to uncontrolled variation in the thickness of the covering over the strainers; that such

variation, whether increase or decrease, would be detrimental to the process, and that the infiltration system, being below the low-water level, cannot be subjected to the same accurate control and careful observations and studies of operation as are possible with a modern filter; furthermore, that the only method of cleaning the beds is by back-flushing, and that such a procedure, without controlling devices for guaranteeing an even distribution of the wash-water, may do decidedly more harm than good; and, finally, that the danger of the introduction of polluted river water into the system was ever present. A special report, dated January 15th, 1909, therefore, recommended that it was unwise to introduce such a system. It is now evident that several of the difficulties foreseen have already occurred; the author's account of the replacements during the summer of 1916 point to this fact.

Messrs. Knowles and Stevenson.

The investigations of the Pittsburgh Filtration Commission in 1907-09, with which one of the writers was connected, showed that no system of filtration, without a sufficient period of subsidence, will effectively treat the Allegheny River water; and that, if the rate of filtration is increased to more than the normal moderate rate of slow sand filtration, a coagulant must be used. The investigations at Cincinnati fixed an average of 125 parts per million as a conservative estimate of the suspended matter in the unsubsided Ohio River water, which could be treated regularly and fairly satisfactorily by slow sand filters. On this basis, a slow sand filter without a period of subsidence would be a failure 65% of the time, if treating this river water.

For all these reasons and because of the many points of inferiority of the Smith infiltration system, when compared with the modern slow sand filter, designed, constructed, and operated in accordance with accepted engineering practice, a negative recommendation on the Smith system was made.

There is, however, nothing new or novel in the Smith infiltration system. Although it was claimed to be an improvement on and substitute for the established method of slow sand filtration, in reality it resembles more nearly, both in principle and in practice, the "filter cribs" which have been used for the supply of water to communities along the Allegheny and Upper Ohio Rivers, and, no doubt, on many other streams, for more than 20 years. In most cases, the construction of the latter consists of a large wooden crib, open at the sides and bottom, placed in an excavation in the river bed, and covered on top with from 5 to 7 ft. of selected sand and gravel.

At the time of the investigations, by the Pittsburgh Filtration Commission, of possible sources of water supply for that city, considerable study was given to the results obtained through filter cribs. Analyses of the river water and of effluents from the cribs indicated

Messrs. Knowles and Stevenson.

that, in general, the effluents from the cribs were clear and free from suspended matter, except at extreme high stages of the river, and that the number of bacteria in the effluents were usually less, but the hardness materially greater, than that of the water from the river itself. The Pittsburgh investigations showed clearly that, early in the life of the cribs, much or all of the water is drawn from the sand and gravel beds lying under and around them; but that, with age, the sand and gravel beds become clogged, and larger and larger quantities of water are drawn down from the river through the top covering. In many cases this downward flow is increased by raking the sand covering, and although the top covering may be broken up in this way or even replaced, it is impossible to clean efficiently the material under and around the crib. With the increased infiltration direct from the river through the top, there is a corresponding deterioration in quality. These statements are fully confirmed by the experience of the numerous water plants with crib supplies which have been obliged to supplement these with filter plants within the past few years.

The author has expressed the opinion that a large percentage of the water drawn is taken from the sand beds, possibly a larger part than that from the open river. This opinion is quite in accord with the experience of the writers, and it would appear not unlikely that during the early life, while the strainers are new, and before the strata at the bottom and sides become clogged, 70% or more of the total water may be drawn from this lower bed. The so-called filtered water, therefore, is in large part a ground-water, similar in chemical characteristics to that which would be had from wells; and in so far as satisfactory results with respect to quality are secured, they are without doubt due more to the quality of the ground-water than to the efficient filtration of the river water. Nor would it be safe either to predict how long the situation might continue, or to expect other than unsatisfactory results with the increase of the proportion of river water. Back-flushing, if forceful enough to be successful, invariably results in disturbing the sand layers to a dangerous degree and frequently forms craters or troughs of washed material, leaving the rest of the bed undisturbed and uncleansed.

It is worthy of note also that, at Parkersburg, the decision between a well supply and a supply by mechanical filtration was a very close one. Either system was considered certain to provide a safe, abundant water supply for the community, and both systems had the endorsement of the best engineering practice and were sanctioned by the most exacting Boards of Health in adjoining States. The system adopted had none of these assurances, and was closely akin to a practice which has been condemned, and is rapidly being abandoned in neighboring States, for water supply for domestic purposes. It would seem that

the time is fast approaching when most of the expenditure of $80 700 will be required to be renewed or replaced by some other system. Messrs. Knowles and Stevenson.

If the story of the development of the Parkersburg Water-Works might be permitted to point a moral, it would seem to be that of the advantage to be gained by every community in having a sovereign State organization, the duties of which should include investigations and experimental studies on all problems concerning water supply and sewage disposal. In this manner a large fund of valuable information might be collected, placing the supervising body in a position to advise local authorities in these matters in an impartial way and with freedom from the pressure of local controversial opinion. Fortunately, this has since been recognized in West Virginia, by the re-organization of the State Board of Health, and the addition to its staff of a competent sanitary engineer. Had the State had such an organization at the time of the adoption of the present water supply system, and had the City been obliged to have that Health Department approve the installation, the community might have been spared the expense of having many different engineers make separate investigations, only to disregard at the end the recommendations of most of them, and to put in a system which was largely experimental and in the adoption of which the theory and practice of water-works engineering was probably not the sole consideration.

the State is now considering when most of the Commonwealth of 1,000,000 must be supplied or undertaken or regulated by some State agency.

The development of the Commonwealth would seem to be wise if the advantages to be secured by a sovereign State organization the duties of which should include investigation and expert control of all problems concerning water supply and sewage disposals. In this manner a large fund of valuable information might be collected, placing the supervising body in a position to advise local authorities in these matters in an intelligent way and with freedom from the influence of local engineering opinions. Fortunately, this has been accomplished in West Virginia by the reorganization of the State Board of Health, and the addition to its staff of a sanitary engineer. Had the State had such an organization at the time of the adoption of the present water supply system, and had the City been obliged to have that Health Department approve the installation, the community might have been spared the expense of having three different engineers make separate recommendations only to disregard in the end the recommendations of none of them, and to put in a system which was largely experimental and in the adoption of which the theory and practice of water works engineering was probably not the sole consideration.

**AMERICAN SOCIETY OF CIVIL ENGINEERS**

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



# THE RECONSTRUCTION OF THE STONY RIVER DAM

## Discussion*

BY MESSRS. J. W. LEDOUX, J. K. FINCH, P. RUTENBERG, FRED F. MOORE, W. S. DOWNS, H. L. COBURN, H. F. DUNHAM, AND ORRIN L. BRODIE.

J. W. LEDOUX,† M. AM. SOC. C. E. (by letter).‡—Mr. Scheidenhelm has written a very exhaustive treatise on the reconstruction of the Stony River Dam. As a result of his work he has formed two important conclusions with which the writer heartily agrees and has been advocating for a great many years, namely, that most masonry dams fail due to sliding, and that too little provision is made for excessive floods. Mr. Ledoux.

Heretofore, the engineer was considered conservative when, on a small water-shed, he provided for a flood of 250 ft. per sec. per sq. mile, but floods of several times that size are likely to occur, and it is only a question of time before they do.

Whether provision should be made for the greatest possible flood that will probably occur at intervals of a calculated number of years involves several questions:

First.—Complete provision should be made, if otherwise lives may be lost.

Second.—If only damage to property and the structure itself is involved, then it becomes a question of whether the cost of making complete provision is greater or less than that of the damage when figured on a present-worth basis. For instance, if a dam is seriously

* This discussion (of the paper by F. W. Scheidenhelm, M. Am. Soc. C. E., published in February, 1917, *Proceedings*, and presented at the meeting of March 21st, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Philadelphia, Pa.

‡ Received by the Secretary, March 16th, 1917.

Mr. Ledoux.

overtopped once every 50 years, causing a wash-out which is estimated to cost \$20 000 to repair, and the cost of making complete provision to prevent that overtopping would be \$10 000, the present worth of \$20 000 to be spent 25 years later at 5% is about \$6 000; so, on this basis, it would be engineering prudence to take a chance, provided no other inconvenience or damage were entailed than the \$20 000. It involves great expense in many cases to make provision to take care of a flood equal to 1 600 sec-ft. per sq. mile. Small water-sheds in the eastern part of the United States, however, are subject to such floods at very rare intervals, and that means a flow corresponding to a continuous rainfall of about 2½ in. per hour.

The writer visited the Stony River Dam a few days after its failure. The following is an extract from his report made at that time:

"The caretaker, Mr. Kerr, who lives with his wife at the house close to the dam, stated that on Wednesday morning, January 14th, about nine o'clock, he noticed water coming through muddy on the down-stream side of the dam at the point where it subsequently failed. By Wednesday night this had become much worse, and on Friday morning at 1 o'clock he notified Mr. Allen Luke, one of the members of the firm of the West Virginia Pulp and Paper Company, that he was afraid the dam was going to break. He was told to open the gates located on the inside of the dam. He stated to his people that these gates could not be opened on account of the stairway leading to them being covered with ice, the temperature being below zero.

"By 2.30 A. M. conditions had become so grave that he notified by telephone the town of Maysville and others located down stream along the banks of the Stony River and Potomac. The water was rushing through the large and increasing space underneath the dam, and by Thursday morning at 9 o'clock the water had fallen in the dam 3 ft. At 9.30, two bays, or 30 ft., broke down, and in half an hour three more failed. By 11 o'clock the water was about 13 ft. below the spillway, and by 6 P. M., only 4 or 5 ft. of water remained in the dam. By Friday morning it was entirely empty.

"The dam was built on a clay formation and under the up-stream toe was a cut-off wall, from 3 to 4 ft. thick. For a distance of 600 ft. or more, on the east side of the valley, this cut-off wall extended down to rock, which, from all accounts, is of an indifferent quality, characteristic of the bituminous coal formation in the vicinity of the Middle Kittanning. The cut-off trench proved to be very difficult, due to its depth, the number of boulders encountered, and water-bearing soil, so that the material sometimes caved behind the shoring and made the work dangerous. The engineers in charge of the work figured that a considerable amount of money could be saved by not continuing on rock, which appeared to be increasingly difficult to reach toward the west end, and so notified the principals of the Paper Company. Not wishing to take any chances, the Company advised a conference with able consulting engineers, which was had, and it was decided to step up the cut-off wall until it had a depth of about 5 ft., and, according to information given me by the Superintendent of the Whitmer Lumber

Company, these engineers concurred in the recommendation of the local engineers that it would be safe to make this change, on account of the quality of the material being very compact clay, sand, and gravel. Mr. Ledoux.

"At the west end of the break, this cut-off wall was not more than 5 ft. deep and about $2\frac{1}{2}$ ft. in thickness. The break extended from Abutment 11 to Abutment 16, and the dam is further damaged to Abutment 19. The stepping up from rock to earth began suddenly between Abutments 15 and 16, the first step being about 11 ft., and at Abutment 11, the bottom of the cut-off trench was more than 20 ft. higher than it was at Abutment 16. It is quite probable that unequal settlement occurred due to the change from rock to earth, as this is where the break took place, and it is hard to understand how engineers familiar with the Austin Dam failure could have consented to any such curtailment of depth of the cut-off trench.

"The writer does not think that the failure of the dam was due to the Ambursen type of superstructure, but it is very clear that, with this type, it is no more safe to economize on the depth of foundation than with any other, because if it is built on an earth foundation and water can flow through under it in material quantities, failure is certain to take place.

"The strongest claim of the advocates of the hollow dam, particularly the Ambursen, is that it is much more economical than the solid gravity type, but, as a matter of fact, on account of its structural complication, with its forms, reinforcement, and necessity of more expensive concrete, it costs not much less for equal stability and utility; but, in order to make the comparison more favorable to their type, they economize in the foundations, which, in the writer's judgment, should be as thoroughly taken care of as with the gravity dam. The case in point is certainly a flagrant illustration of their bad judgment in this respect.

"According to the plans of the dam revealed in a paper presented by Mr. Bayles in *Engineering News*, of January 22d, 1914, it is going to be a very difficult matter to decide just how far to carry the repairs so as to prevent subsequent failure of some other portion.

"It is the practice of conservative engineers never to build a masonry or concrete gravity type of dam unless they can found it on rock for practically the entire distance, and, in addition thereto, it is the practice to carry a cut-off wall down far enough underneath the main foundation to be practically impervious to water. The writer does not see what advantage the Ambursen type of construction possesses which makes this requirement unnecessary, but, in the case of the Stony River Dam, none of the main structure was founded on solid ledge rock and only two-thirds of the cut-off trench.

"Fortunately, no lives were lost, but that was probably due to the gradual emptying of the dam. If the break had taken place suddenly, when the dam was full, like that at Austin, Pa., it is probable that there would have been some loss of life, although the towns were far enough down stream so that they could be notified by telephone at least an hour before the water reached them.

"From an inspection of the construction, which is very readily made from a concrete walk, running longitudinally within and throughout

Mr. Ledoux. the entire length of the dam, the writer is convinced that the superstructure was built in a workmanlike manner, but the concrete appears to be somewhat softer than it should, according to the best practice. On account of the large amount of snow all around and the débris in the valley on the down-stream side of the dam, and the ice on the up-stream side, it was impossible to make any close inspection of the soil or the geological formations.

"In one of the bays are several gates for letting off the water in case of emergency, or for any other reason, and there is also in an adjacent bay a large gate for the same purpose. An archway entrance goes through the wall between these two bays. At the side of one of them is a concrete stairway and an iron hand railing, and this, at the time, was covered with ice, due to drippings from a crack in the up-stream portion of the dam, making the valves inaccessible. A casual inspection, however, showed that this ice could have been chopped off with a hatchet or axe in a few minutes, or poles or a ladder could have been run down from the concrete walk to the floor where the valves were located in the adjacent bay. If these gates had been opened wide from Wednesday morning, it is quite probable that the water in the dam would have been so lowered as to prevent failure. It is better, however, for the engineering profession, that the dam did fail, as it will further impress upon their minds the necessity of care and thoroughness in the design and construction of dams in general.

"The nature and abundance of suitable soil and the depth of the bed-rock indicate that the proper type of structure would have been an earthen dam with a concrete spillway on either the east or the west side-hill, and this could probably have been built safely at as low a cost as the type that was adopted."

Mr. Finch. J. K. Finch,* Assoc. M. Am. Soc. C. E. (by letter).†—When the failure of the Stony River Dam was first reported, and before complete details were available, the writer pointed out‡ that dams of this type were particularly weak as regards sliding. Articles on the inclined slab and buttress dam always emphasize the great stability of these dams against overturning, but fail absolutely to call attention to their weakness as regards sliding. When the buttresses extend down to rock, the rock surface should be roughened and the buttress well bonded to it. When the dam is founded on a spread footing, four important points must be considered.

1.—The material just below the floor of the valley, on which the spread footing between the buttresses is placed, must be of a satisfactory, compact, substantial character that will not "flow" even when wet.

The idea has seemed to be prevalent in the minds of some designers that the slab and buttress dam of this type can be set down anywhere, on the surface of almost any kind of ground. In a pamphlet advertising a dam of this type, it is described as built on "a foundation

* New York City.

† Received by the Secretary, March 8th, 1917.

‡ In a letter to *Engineering News*, January 22d, 1914, p. 202.

of quicksand and hay." Without knowing the details, it is difficult to criticize the design, but the writer hardly considers it as conservative engineering. This particular type is advantageous of course from the standpoint of economy, when the depth to rock is great, but the foundation material is certain to be wet at times, and a soft rock, hardpan, compact gravel and, in certain cases, a compact clay would seem to be the foundation range for a dam of this character. Mr. Finch.

2.—The unit pressure on the bottom of the spread footing should be low and uniform, and the possibility of upward pressure on the footing, or of back-wash undermining it, should be guarded against.

If the slab is inclined at the usual angle of about 45°, uniform pressure at the base of the buttress can be secured by making the base about 1.3 of its height. It is usually assumed that the entire weight of, and pressure on, the slab is transmitted to the foundation through the buttress. A small "wash-wall" at the down-stream edge of the dam prevents undermining of the footing and also confines the foundation material, but, in the writer's opinion, this should not be counted on to resist sliding. Weep-holes are provided in the footing in order to eliminate upward pressure on the base.

3.—The cut-off wall should extend down to impermeable material, below any seams which might communicate between the reservoir floor and the stream bed below the dam.

Practically all the failures of these dams have been due primarily to inadequate cut-off walls. Note especially the Pittsfield, Mass., Dam and the less fortunate Stony River Dam. It is said that engineers should profit by the expensive experience of others, but, unfortunately, some engineers are still willing to "take a chance", usually, however, at the expense of others. It is also far from rare to find cut-off walls built without reinforcement in soft material requiring sheeting and where the sheeting is subsequently pulled. It would assuredly seem that all cut-off walls which are not built in compact earth in an unlined trench, or in a trench in which the sheeting is left in place, should be reinforced.

4.—Stability against sliding must be adequately provided for.

In the letter previously mentioned the writer said:

"A simple calculation shows that in the case of the Stony River Dam a coefficient of friction of about 0.52 is required to prevent sliding. * * * the soil at the dam site [is described] as 'a yellow clay mixed with fine sand or gravel which is underlain by a stiff blue clay, and some seams of a black material occurring in places.' At the best, this material could not be counted on to give a higher coefficient than 0.70 to 0.80 and when wet might give as low as 0.35. Moreover, the design shows weep-holes in the spread flooring, indicating that the designer expected water might, as it doubtless would, occur under the base. Under these conditions it is difficult to see what factor of safety there was against sliding except the shearing value of the

Mr. Finch. concrete of the cut-off wall (which in this design is unreinforced) and the small 4-ft. wash wall at the lower edge of the floor that should most certainly not be counted on as offering any resistance."

Lack of stability against sliding is a well-recognized point of weakness in reinforced concrete structures of the retaining type, and must always receive careful consideration. In retaining walls, it is provided for by projections on the base, and, in the case of a dam of this type, Mr. Scheidenhelm has described the various methods available, and has finally selected the method of utilizing the upper portion of the cut-off wall for this purpose. The writer has been advocating the same method of design in teaching a class in reinforced concrete, and is pleased to see this feature treated so completely and thoroughly in this paper. The slab and buttress dam, properly and conservatively designed, offers economic advantages which undoubtedly make it a most desirable type in certain locations; Mr. Scheidenhelm's paper clears up one of the important points regarding its design, which has long been neglected, and is a most valuable contribution to the literature of the subject. It is indeed unfortunate that the West Virginia Pulp and Paper Company, which, like many other concerns, did not try to economize at the expense of safety, but, on the other hand, insisted from the first on a reliable structure, should have been, through no fault of its own, the victim of such an unnecessary and expensive experience.

One other point in the design of these dams remains to be settled, namely, their architectural treatment. A design* for a dam at Coatesville, Pa., by Alexander Potter, Assoc. M. Am. Soc. C. E., illustrates the only attempt, so far as the writer is aware, to design one of these structures of really pleasing appearance. In the Stony River Dam, as the author pointed out, no such treatment was considered necessary, but it is more and more evident that, in engineering structures which are near cities or much traveled highways, the appearance must be considered. Too often work of this kind is attempted by a designer who knows nothing of the fundamental principles involved, or it is left to a young architect who is familiar only with house architecture and attempts to apply it to a totally different structure, or applies some inappropriate form of ornamentation that detracts from, rather than enhances, its appearance.

The Engineering Profession lacks in any "tradition", regarding the proper treatment of engineering structures, such as the architect has in the treatment of buildings and on which he is prone to lean, as many believe, too heavily. Many of the engineering works in Europe, which have been held up as examples of good engineering architecture, are merely house architecture applied to engineering

* *Engineering News*, February 26th, 1914, p. 461.

work. The true solution of the problem is in developing a suitable treatment, forming, for engineering structures, a tradition based on the same fundamental elements as those on which architecture (building composition) is based, but emphasizing the structural features of the engineering design to a much greater extent than is common in architecture. Mr. Finch.

The fundamental considerations of unity of effect, balance, proportion, harmony, and climax cannot be taken care of after the structure has been planned and partly designed. They must be kept in mind from the very first and receive their proper consideration in the preliminary schemes, sketches, and plans for the work. The only man who can do this is the engineer himself, who realizes the structural and economic principles involved, and, it is hoped, may soon come to realize the necessity for proper architectural composition as a basis for a final and simple decoration in which the structural features should be emphasized.

P. Rutenberg,* Esq. (by letter).†—European engineers have learned much from their American colleagues, and the war proves the extent to which American methods of solving technical problems, of their organization and realization, have been adopted and developed in Europe. In the special field of dam building they have heard and learned from such world-famous works as the Croton, the Roosevelt, the Elephant Butte Dams, and many others. Mr. Rutenberg.

If the writer now takes the liberty of criticizing the construction of the Stony River Dam, it is because this structure, in his judgment, is lacking in just that efficiency which is characteristic of American engineering.

*Foundations.*—As described by the author, the foundation soil at the site of the dam is mainly of a clayey nature, heterogeneous, pervious, and of poor bearing value in general. The examination of the old cut-off wall, and of the buttress footings remaining intact, showed that abundant percolation and leakage existed in the foundation soil of the original dam. "The contact between the original footings and the foundation soil was in some places found to be faulty. * * * Hollow spaces were found under the footings around the 4 by 12-in. weep-holes." In some bays, it was discovered that "the foundation soil had been washed away at the junction between the footings and the cut-off wall. * * * The construction joint would naturally open somewhat, due to the deflection of the foundation soil when loaded. * * * Observations showed that grout under pressure traveled under the footings for a distance of at least 100 ft.", etc. (pages 241-242).‡

* Petrograd, Russia.

† Received by the Secretary, March 21st, 1917.

‡ *Proceedings*, Am. Soc. C. E., February, 1917.

Mr. Rutenberg. All this was produced by the leakage under pressure during 65 days only, the period in which the original dam functioned, until it failed.

The faulty conditions of the foundation soil were remedied by pressure grouting through the original weep-holes. In this way, the original foundation soil under the footings was more or less compacted, but there surely remained other hollow spaces and pockets from which water under pressure passed, not to the weep-holes of the footings, but in other directions of least resistance, not observed in the reconstruction of the dam. It will be shown later that the author himself admits it, and that his statement (page 242) that "the foundation soil under the original footings was thoroughly compacted so as to warrant the assumption that the footings are everywhere in contact with the foundation soil", is not entirely correct. Percolation and leakage will cause harmful and dangerous uplift pressure in any dam, with any foundation soil, even the best.

The uplift pressure applied to the up-stream part of the base increases the overturning moment of the dam and the unit stresses in the structure and on the soil at the down-stream foot of the dam, where, under the full load of the dam, the values of these stresses are maximum. The uplift pressure on the up-stream part of the base decreases, consequently, the resistance of the dam to overturning, and may render insufficient the resistance of the structure and that of the soil at the base to the efforts of compression and shearing developing there. The same conditions created by the uplift pressure decrease also the resistance of the dam to sliding. This has been the cause of the failure of many dams, even with good foundation soils which have possessed the best frictional resistance.

Therefore, it is impossible to state, as Mr. Scheidenhelm does (page 201), that "the smaller the coefficient of frictional resistance, the less the net effect of the uplift pressure."

Consequently, it is clear that percolation and leakage must be eliminated by all possible means in any dam, in general, and in the Stony River Dam, with its heterogeneous and pervious clay foundations, in particular.

The interception of the leakage and percolation in this dam should have been made directly and immediately on the up-stream face of its cut-off wall and discharged down stream by special channels without any pressure.

Now, the principles on which the Stony River Dam was reconstructed are quite different. On page 251 we read:

"It was believed to be preferable, therefore, to take such risk as might result from the existence of pockets, as it were, of unrelieved uplift pressure near the down-stream side of the cut-off wall rather

than to invite further leakage by tapping such relatively harmless, deep-lying pockets of water under pressure." Mr. Rutenberg

This statement, it seems to the writer, does not tally with the most vital principles in dam-building, namely, that water under pressure in the foundation soil is never harmless.

According to the foregoing consideration, the system of drainage adopted for the Stony River Dam consists of 3-in. wrought-iron pipes, perforated with four ⅜-in. holes per linear foot, one hole in each quadrant; three pipes per bay, driven into the foundation soil vertically, the first "about 20 ft. down stream from the cut-off wall, except at the bays of lesser height, where they are at a minimum distance of 12 ft. from it."

The writer believes that:

1.—A 3-in. pipe with four small perforations per linear foot for a bay 15 ft. high is useless for drainage, even if the drainage pipes described by the author function perfectly well.

2.—The small perforations in the pipes must undoubtedly have been clogged with clay in the process of being sunk. Mr. Scheidenhelm states (page 252) that "any considerable hydrostatic pressure near the drain pipes would soon open the perforations." He admits the presence in the foundation soil of considerable hydrostatic pressures at a distance of 20 ft. from the down-stream side of the cut-off wall, which is about the middle of the base. Consequently, by the improper location of the drainage pipes, alone, the up-stream half of the base is exposed to dangerous uplift pressures, with all previously stated consequences, *viz.*, decrease of resistance by overturning, increase of the unit stresses in the structure and on the foundation soil, saturation of the clay foundation soil with water, and decrease of its resistance to sliding; which, as the writer understands, cannot but be harmful, and all this solely because the drainage pipes, even assuming that otherwise they function perfectly well, are not in the proper places.

3.—Some of the drainage pipes have been driven vertically to a depth of 20 ft. into the foundation soil. When they are filled with drained water, it is clear that their lower perforations are submitted to the pressure of a column about 20 ft. high, and at this lower stratum of the foundation soil the leakage will not be able to penetrate into the drainage pipes, unless it is of greater hydrostatic pressure. Only a small part of the leakage will penetrate through the small perforations; the remainder will continue its way down stream into the foundation soil, where the resistance is weaker. Besides, there is a possibility that the drainage pipes will be filled through the higher perforations from the higher strata of the foundation soil, and that the collected water will be conducted through the pipes and their

Mr. Rutenberg. lower perforations, by the water pressure thus formed, into the otherwise dry lower strata of the foundation soil.

In fact, on page 254, the author takes cognizance of such phenomena, for which manifestation he "has been unable to advance any explanation which is satisfactory to himself."

The writer, therefore, considers these pipes, not as a drainage system, but rather as a means of introducing water into the foundation soil, and not only useless, but dangerous to the safety of the dam.

*Resistance to Sliding.*—On page 162, we read:

"The method adopted [for increasing the resistance to sliding] is believed to be new, and consists in the use of anchoring walls extending to a considerable depth into the underlying foundation soil, and, in effect, utilizing the weight of that underlying soil as well as the resistance (to horizontal movement) of the soil immediately down stream from the structure."

These anchoring walls are two: one at the heel, and another at the toe, of the original structure which remained intact.

To render the above-mentioned service, the heel-wall must be strong enough to be able, in case of a down-stream sliding of the dam, to function as a cantilever which can overcome the resistance of the underlying foundation soil to sliding and shearing.

The paper does not give the dimensions and calculations of the anchoring walls, but, from Plate V, it is clear that they are too weak to serve this purpose. The heel-wall is not able to compact the underlying foundation soil, and, therefore, make it less pervious by any down-stream movement on the part of the dam, as the author believes. It will simply be broken in this case, opening the joint between the upper structure and the cut-off wall, as happened in the original structure, according to the description on page 213.

The toe-wall, as constructed, will aid in collecting the leakage under the dam. Consequently, it will increase the uplift pressures and the quantity of water in the foundation clay, decreasing the resistance of the dam to overturning and sliding.

Openings should have been made in the vertical part of this wall, directly under the footing floor of the dam to provide a free exit for the leakage.

As far as the two anchoring walls are concerned, the writer does not agree with the author's assertion that "the reconstructed dam acts essentially as a monolith" and, consequently, he does not agree that "the horizontal member of the toe-wall in effect increases the width of the base of the dam, thus decreasing the unit vertical load on the foundation soil." (Page 214.) The efforts transmitted by the buttresses to the foundation soil have their maximum value at this point, and, the writer believes, under the existing circumstances, will rather break this horizontal member of the toe-wall.

The under-cut slope of the heel-wall presents, besides, a surface for the application of uplift pressures, which, with their big moment arm, decrease also the stability of the original structure against overturning. Mr. Rutenberg.

In the reconstruction of the intact part of the dam, the only things to do were: to fill, if possible, all the holes found in the foundation soil; to repair the cut-off wall, and to build a reinforced concrete wall up stream from the old cut-off wall, bearing upon this by concrete apertures, in order to form chambers which would be able to maintain the percolation through this drainage wall and conduct it without pressure down stream from the dam by convenient channels.

The two anchoring walls with the drainage pipes have weakened the original stability of the dam.

The "new theory" of resistance to sliding, with the investigation about the "plane of least resistance", with the tests, calculations, and deductions of an "average" coefficient of frictional resistance of 0.61 in wet clay, is an interesting theoretical exercise, but without practical value for the reconstructed dam. One is the frictional resistance of a cubic foot of clay, pulled by two men under the conditions of the improvised laboratory described; the other is the effective frictional resistance of the wet clay of heterogeneous structure in the real foundation soil loaded by a head of 90 ft. and more, or by about 6 000 lb. per sq. ft.

There are dams in which the necessary weight is obtained by clay, sand, pebbles, even water, included in a reinforced coffer, the bottom of which constitutes an elastic monolith with other coffers. If percolation took place, it would be applied to the bottom and, consequently, to the whole structure of the dam. The material of light weight cannot be washed from them.

In the Stony River Dam, however, engineers cannot seriously consider for its resistance to overturning and sliding, the "weight" of the underlying foundation soil, which can be and is, as is known, washed away by the existing leakage and percolation.

*Spillway.*—Not being acquainted with the hydrological conditions of America, the writer cannot give an opinion as to whether the spillway capacity provided in the reconstructed dam is adequate or excessive; but one thing is clear: that by adding 3½ ft. of water to the original level in the reservoir, the horizontal water pressure and its overturning moment are increased by about 20% for the typical section indicated on Plate V. The increase is greater for the sections of less height.

The parapet-like addition to the original structure and the curtain-wall and roof, contribute a vertical effort of important value, which passes into the down-stream third of the base. Both conditions increase the overturning moment, consequently, they also decrease

Mr. Rutenberg.

the safety of the original structure. The upper structure in the new spillway is made heavier than in the original dam.

For each horizontal section the water loading is exactly the same in the new and in the old structure. Thus, either the original structure is insufficient or the excess of the new is useless. In any case, the subjection of the original structure to greater loadings and unit stresses than those for which they were evidently designed is dangerous; 22 000 lb. per sq. in. = 15.5 kg. per sq. mm. in the steel, and 800 lb. per sq. in. = 56.5 kg. per sq. cm. in the concrete, are too much, at least, according to European provisions. The paper does not give the necessary data with which it would be possible to compute the unit stresses in the structure and on the soil, but the writer is sure that the load on the soil at the down-stream foot of the dam is excessive. Since, under favorable circumstances, a part of the original structure remained intact, it was advisable to maintain the original maximum water level.

The allowed over-topping of 2½ ft. of water from a height of 40 ft. between Buttresses 19 and *A* is, it would seem, also dangerous. The shock of the falling water will not only break the provided horizontal and inclined reinforced mat, but, when that occurs, it will wash the soil from under the dam footings.

On page 171, Mr. Scheidenhelm, criticizing the rupture of the original dam, says: "The conditions obtaining at the Stony River dam site are such that a safe dam could be built there. * * * The dam should, and could, have been designed and constructed so as to be absolutely safe."

The writer believes the reconstructed dam is less safe than the original structure.

The flaws in the foundation soil which had not been taken care of by the constructors of the original dam, widened and became more pronounced by the water pressure formed by the old dam. After the rupture occurred, some of these flaws came to light and could not but be remedied; so that the foundation soil was relatively reinforced, thus giving it another short lease of life, longer than for the original dam.

The writer is firmly convinced, however, that were the reservoir now to be emptied and the foundation soil examined it would be found to be in a very bad state.

"Certain of the drainage openings yielded muddy water, instead of the normally clear water. * * * Such openings were grouted shut, rather than take any chances of harmful erosion under the footings." (Page 254.)

This "harmful erosion" has surely found other ways, unobserved by the author.

The writer is firmly convinced that the reconstructed dam will also be broken, perhaps soon, if adequate repairs are not made in time. It is a question of preventing a new disaster. These are the writer's reasons for taking the liberty of criticizing the work done in the reconstruction of the Stony River Dam. Mr. Rutenberg.

FRED F. MOORE,* M. AM. SOC. C. E. (by letter).†—After study of this excellent description of the interesting reconstruction of an uncommon type of dam, two impressions stand out prominently: Mr. Moore.

First, that there is a dearth of information regarding sub-surface conditions as a guide to the original designs. Even now, comprehensive knowledge of the character of the foundation materials, and the behavior of these materials under the conditions imposed, does not appear to be sufficient to justify the conclusions reached by the author as to the margins of security of the reconstructed dam. Tests of frictional resistances of soils, as carried out in connection with this work, are of doubtful value as an aid to the judgment, because of the small areas in contact, and the values of frictional resistance adopted for the design seem to be considerably larger than even the indications of these experiments would justify.

Second, with reference to the function of the walls at "heel" and "toe" in resisting movement of the structure as a whole, the analyses which consider sliding and overturning independently would appear to befog rather than to aid the judgment. Materials such as these, mixed with clay and saturated with water, perhaps under considerable pressure, and subjected to large loads, would flow, or, at least, transmit pressures in all directions like a liquid. In order that the cut-off wall at the "heel" may aid in resisting sliding as described, there must be a sensible movement of the structure as a whole, with consequent large uplift on the base of the dam. Hence, the sliding and overturning, or, perhaps better, the floating, of the structure must be considered with reference to the effect or reaction of these tendencies one on the other. That is, the materials under the dam, with reference to the function of the cut-off wall in resisting sliding, must be considered as a flowing medium rather than one in which a sliding may occur on certain planes like one solid on another.

The wall at the "heel" does not appear to have sufficient strength for the purpose contemplated. If this wall is fractured, water under reservoir pressure in considerable quantity would have access to the materials under the dam. The drainage provisions appear to be a doubtful reliance for minimizing uplift pressure. The danger of portions of the dam being floated off the foundations from leakage of water under pressure through the cut-off at the "heel", and through,

* New York City.

† Received by the Secretary, March 21st, 1917.

Mr. Moore.

or under, the cut-off at the "toe", at a time of large overflow, appears not to have had sufficient consideration. Although the idea of using the materials between the cut-offs and immediately down stream from the "toe" to resist sliding is a good one, it is doubtful if the reconstructed dam has a sufficient margin of security to warrant a conclusion that it is not likely to fail again.

The run-off assumed for the design of the spillway appears to be reasonable for the purpose, but the margin of 6 in. against overtopping is too small. A flow of 2 ft. of water over the intermediate spillway would not fall "well-nigh" vertically. Such a flow would strike the slope of the fill (Plate V), which is inadequately protected for such a condition. Even water falling into the concrete channel from the top of the dam would start serious trouble through erosion of the bank by wash. Regardless of damage by falling water, the concrete channel is too small. Any considerable flow over the intermediate spillway would be almost certain to start cutting of the materials at the "toe".

The use of brittle steel pins for the flash-board support is unusual, and would seem to be an expedient of doubtful value.

Mr. Downs.

W. S. Downs,* Esq. (by letter).†—Ignorance of the coefficient of sliding friction between rock sub-strata, and the resulting neglect to provide the proper safeguards, have been the cause of numerous dam failures. Such failures never occur where the rock is of igneous origin, such as granite, trap, or the like, but authenticated records show that this danger lies in the softer sedimentary formations where the rock is horizontally stratified or nearly so.

Probably in no extensive geological formation is sliding or slipping of materials more in evidence than in the formations of the Carboniferous period, especially that portion of these formations which include the coal measures of the Appalachian fields and in which the Stony River Dam is situated.

These formations are highly stratified, the strata usually consisting of rather soft shales, ranging from a very soft fire-clay shale to a sandy shale, with occasional thin strata of coal and limestone and lentils of sandstone. One of the chief difficulties in studying the formations, however, is the non-uniformity and non-persistency in the character of the strata. The same stratum sometimes varies from a hard sandy shale to a soft fire-clay shale within a few hundred feet. The shales decompose and disintegrate very rapidly when exposed to atmospheric conditions, and, when decomposed, the resulting clays and soils are difficult to drain and exceedingly unstable.

* Morgantown, W. Va.

† Received by the Secretary, March 21st, 1917.

The available data on the coefficient of static friction, especially as to layers of shales and clays, are extremely meager. In fact, very little has been learned on this subject since Morin made his experiments in 1834. The intensity of the pressure and the length of the time of contact between the two surfaces are factors the influence of which on the universal laws of friction are but vaguely understood. Therefore, until more extensive experiments have been made, and these general laws are more thoroughly understood, it behooves the engineer not only to make as thorough tests as practicable on the materials in question, but to use an ample factor of safety in designing. Mr. Downs.

Unfortunately, tests on a large scale are practically impossible, and engineers must often base their conclusions on tests of small areas of contact and with relatively light pressures. In applying these tests to the whole foundation area, they must depend largely on their knowledge of the geological conditions, coupled with an abundance of good judgment.

Mr. Scheidenhelm's tests on the frictional resistance of the foundation soils, as shown in Table 2, are interesting, and are valuable as an indication of the character of the materials in question. However, too much dependence should not be placed on the results of these tests. The same materials might show very different results when subjected to the more intense pressure of the dam for an indefinite length of time; and, moreover, the condition of saturation, as it must exist beneath the dam, might change the value of the coefficient of friction very materially. The writer has had occasion to study numerous slips and landslides in materials of this character, and has been surprised to note the vast difference in frictional resistance between dry and wet clays. He has in mind one instance where dry clay soil indicated a coefficient of frictional resistance of more than 0.7, but after it had become thoroughly saturated for a considerable time, the coefficient of frictional resistance was less than 0.2.

Referring to the results of Mr. Scheidenhelm's tests, in Table 2, it will be observed that there is considerable difference in the frictional resistance between wet and moist samples of the same materials, and therefore the question arises: Might not the frictional resistance be still lower than is indicated by the tests when the clays are subjected to the conditions that will exist in the foundation materials of the dam?

At first thought, the coefficients of frictional resistance for shales (0.40 and 0.50), assumed by Mr. Scheidenhelm in the design for the reconstruction of the dam, may seem to be rather low. However, it should be remembered that the shales in question have "pronounced horizontal laminations" and are somewhat disintegrated, and that clay seams exist in places.

Mr. Downs. The lamination planes in the fire-clay shales of the coal measures often show a slick, smooth surface, sometimes of a more or less slippery nature. It is very evident that the cohesion between the substrata at these planes must be exceedingly slight, and, unless the planes of contact are very much warped or of limited extent, the resistance of the materials to sliding must be rather low.

Mr. Coburn. H. L. Coburn,* M. Am. Soc. C. E. (by letter).†—The Engineering Profession is to be congratulated that there are engineers who have the time, the disposition, and, more especially, the ability, to prepare such a paper as this. One rarely sees a more complete exposition of an engineering problem from inception to finish than is here presented. The writer says from inception, for, to all practical purposes, this was an entirely new problem, as few or no data were available for the author's use.

As originally built the Stony River Dam was designed to meet certain stated conditions of foundation, which, subsequent events proved, did not apply, and the failure of the dam is attributable to this fact alone and not in any way to the type of structure. No dam designed to fulfill the conditions given could have withstood the treatment to which this one was subjected, and the writer thinks it an evidence of sound fundamental principle that the dam failed only in part and that part so gradually.

This is neither the time nor the place to discuss the conditions that led to the designing and building of this dam on data that were wholly inadequate and erroneous. The great lesson to be learned from the failure is the advisability, not to say necessity, of complete preliminary investigations on which to base design, and adequate and competent engineering supervision of construction—money thus spent is "well spent."

As to the reconstruction, though the writer doubts that all the safeguards against a possible second failure, which Mr. Scheidenhelm has taken, were necessary, yet he feels that, in the circumstances, they may be justified. If more accurate data were available as to "passive thrust" or resisting power of the local soils, and more real information as to coefficient of friction between these soils and the concrete cast thereon, engineers might criticize his details in some respects, but with the limited knowledge of actual values, and particularly in view of the fact that this was a case of rebuilding a structure which had partly failed, and about which very sensational stories had been circulated, the writer considers that if the dam was to be rebuilt at all, it was good judgment to assume the "worst possible conditions" and to make assurance doubly sure.

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* New York City.

† Received by the Secretary, March 21st, 1917.

H. F. DUNHAM,* M. AM. SOC. C. E. (by letter).†—In the discussion of a paper on the repairs made to a structure, it may not be in order to refer to the reasons for erecting that structure; but, if such reference is allowable, replies to two or three questions may be of interest. Mr. Dunham.

An article published in a technical journal‡ at about the date of the construction of the Stony River Dam, contained a few words relating to the object to be secured by the improvement. At some point on the North Branch of the Potomac below its Stony River tributary there was a paper mill with an insufficient supply of wash-water in dry-weather periods. There was no mention of the use of an increased flow of the river for power purposes. There were no figures for the dry-weather discharge of the main stream at the mill, nor of its tributary. The additional quantity of wash-water needed was not given, and, furthermore, there was no longitudinal profile of either the North Branch or the Stony River, and no description of the river valleys or reference to any other method of providing the unestimated but desired supply.

It should be generally admitted that if, within a few years, the supply of wash-water has approached the desired quantity there would certainly be enough at all times, if the natural or original conditions pertaining to those streams could be restored. When the entire district was heavily wooded, the winter snows were longer in melting, the ground did not freeze to such depth as now and more of the spring rainfall remained as ground-water for a longer period. Moreover, when floods were less frequent, fallen tree trunks and branches lodged in narrow channels, holding the water back. The beaver built his dams, with similar results, and when they were old he, too, repaired them. All of this was favorable to a longer and more uniform flow of water in dry-weather periods than obtains at the present time. It may be within reason to inquire how far Nature's regulation of flow in such streams can now be imitated. The forest and frost are items that cannot be changed, but, given favorable topography and soil, the fallen timber dams and those made by the beaver with the higher ground-water table could be copied successfully and possibly improved.

The photographs in the paper, and the dam itself, 1 000 ft. long at the most favorable location, indicate a valley of some width. The sandstone mentioned, and the fact that the water found its way under the cut-off wall, show the existence of a somewhat porous soil. It is evident that the structure described was expensive. Had a part of that expense been used to construct a number of low but permanent dams at favorable places for low dams, each to be provided with a

* New York City.

† Received by the Secretary, March 21st, 1917.

‡ *Engineering News.*

Mr. Dunham. cheap low-water sluice-gate that would require a minimum of attention, a considerable change in the dry-weather flow of the stream would have been secured. The author refers to the existence of ground-water in and about the foundations of the dam. To raise the level of such ground-water over considerable areas gives available storage, and water thus stored is not subject to such rapid evaporation as surface water.

The dams, of course, would hold back surface water as well as ground-water. The extent to which the flow of a stream is regulated by higher ground-water level is often surprising. This is well illustrated by rivers flowing through wide flood-plains which, sponge-like, retain large quantities of flood water which are gradually returned to the river in its lower stages.

At certain seasons, especially in the spring, streams of moderate or sluggish current may carry too much silt to be suitable for a supply of wash-water. In the article previously referred to no mention was made of any filtration process, but, if that method is used, a reservoir below the mills suitable for the purpose of sedimentation and refiltration should have substantial value.

The writer has no wish to "go behind the returns" or to ask for any reference to the business affairs of business men that need not be disclosed. The first studies pertaining to any improvement are always interesting, and they are quite apt to be overlooked in the description of a finished work.

How the cost of the dam as it is now compares with the estimated cost of a gravity-section dam of the same general dimensions would be of interest, as would also the question of a curved versus a straight dam for structures of either type in that place.

Mr. Brodie. ORRIN L. BRODIE,* M. AM. SOC. C. E. (by letter).†—The author is to be thanked for the lucid and well-arranged presentation of the subject matter relating to the reconstruction of the Stony River Dam, and the thorough manner in which he has treated the various phases of that work.

One of the most striking features of his design, to the writer at least, was the unique method of anchoring the up-stream heel and cut-off. Relative to this matter, certain statements of the author in his brief outline of the previous failure of the structure were impressive. These were:

"Failure occurred where the up-stream cut-off wall extended only a short depth (5 to 7 ft.) into the over-burden."

"Failure was caused by undermining due to leakage."

"The type of dam was not in any way a cause of failure."

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* New York City.

† Received by the Secretary, April 4th, 1917.

Mr. Brodie.

Thoughtful consideration of these three statements taken together forces on the writer the following query:

Would not a dam subject to considerable head of water, such as the one described, but containing sufficient mass that could contribute by its dead weight a resistance *per se* to sliding and overturning tendencies, be more satisfactory in every respect than one of the type here considered? Even though the hollow concrete type, such as the one described in the paper, may possess decided advantages in the matter of stability against overturning, there is always the possibility of the admission of water below it, which, besides increasing its tendency to slide, destroys any advantage inherent in its form in the matter of stability. Besides, as shown by the author, an up-stream cut-off thoroughly anchored to the heel seems to be a vital necessity. It is also significant that the reconstruction involved a quantity of masonry equal to that of the initial construction, which might better have been utilized in the first place.

The method of computing the storage effect of the reservoir area with respect to the spillway run-off was interesting to the writer, as was the detail of the spillway shape, as depicted on Plate III and noted by the author as being a more advantageous crest than that of the old spillway.

An investigation as to spillway run-off with respect to two basins, the waste weirs of which were at different crest elevations, was made by the writer, and involved an interconnecting tunnel between these two reservoirs (constituting the substitute supply works, connected with the new Kensico Reservoir for New York City).

The understanding of the method will be facilitated by the following nomenclature:

Volumes are in cubic feet; flows in cubic feet per second; and times, in seconds.

$H_1'$, $H_2'$, and $H_1''$, $H_2''$, are elevations of water surfaces in the respective reservoirs, Nos. 1 and 2;

$t$ = time for Reservoir No. 1 to rise through a small distance ($H'$ to $H''$);

$V_1$ = increment of capacity due to $(H_1'' - H_1')$ for Reservoir No. 1;

$V_2$ = increment of capacity due to $(H_2'' - H_2')$ for Reservoir No. 2;

$I_1$ = average inflow during time, $t$, from water-shed of Reservoir No. 1;

$I_2$ = average inflow during time, $t$, from water-shed of Reservoir No. 2;

$i$ = average flow, during time, $t$, through tunnel and from Reservoir No. 1 to Reservoir No. 2;

Mr. Brodie.

$Q_1'$ and $Q_2'$ = flow over respective weirs at the beginning of time, $t$; and

$Q_1''$ and $Q_2''$ = flow over respective weirs at the end of time, $t$.

For Reservoir No. 1:

$$t = \frac{V_1}{I_1 - i - \frac{1}{2}(Q_1' + Q_1'')} \quad \text{.................(1)}$$

For Reservoir No. 2:

$$t = \frac{V_2}{I_2 + i - \frac{1}{2}(Q_2' + Q_2'')} \quad \text{.................(2)}$$

Eliminating $i$ between Equations (1) and (2):

from Equation (1) $\quad t\left[I_1 - i - \frac{1}{2}(Q_1' + Q_1'')\right] = V_1 \text{.......(3)}$

from Equation (2) $\quad t\left[I_2 + i - \frac{1}{2}(Q_2' + Q_2'')\right] = V_2 \text{.......(4)}$

from Equations (3) and (4)

$$t = \frac{V_1 + V_2}{I_1 + I_2 - \frac{1}{2}(Q_1' + Q_1'' + Q_2' + Q_2'')} \quad \text{.........(5)}$$

Method, by successive approximations, of applying the foregoing expressions:

$V_1$ is a direct function of $t$ and $i$, that is, $V_1 = f\ (t, i)$ and must be so considered.

Begin with Equation (5): First, assume values for $V_1$ and $V_2$ and corresponding values for the other terms in the right-hand member. Substitute the value of $t$ thus found in the left-hand members of Equations (3) and (4), with the corresponding new value of $i$, until Equations (1) and (2) are satisfied. The second value of $t$ can thus be tested by Equations (1) and (2). The values of $V_1$ and $V_2$ of Equations (3) and (4), respectively, must agree with the values of $V_1$ and $V_2$ in Equation (5).

The calculations, as briefly outlined in the foregoing, may be made in connection with the use of chosen run-off diagrams, together with capacity curves prepared both for the weirs, tunnel, and reservoirs. The results may be shown by appropriate curves of reservoir rise with time intervals from the beginning of the storm, or with respect to any stage desired.

In connection with the consideration of the spillway shape, the writer has determined the applicability of a parabola, the origin of

which may be taken at the actual crest of the weir and the parameter at about 1.8 times the head of full reservoir level on the so-called "theoretical crest" (that is, the actual crest of a corresponding thin-edged weir). This parabola, with axis vertical and through the crest, will determine approximately the lower nappe or sheet of overfall for a weir. Mr. Brodie.

The study leading to this conclusion was based on weir experiments by M. Bazin. This parameter may be increased for practical considerations to as much as 2¼ times the head.

The writer is not aware of the detailed method by which the shape of the new spillway crest of the Stony River Dam was fixed, but, by the foregoing method, he calculated the crest for the given head and was interested to find that, as far as could be ascertained by careful scaling of the spillway of Plate III, a practical identity resulted. The advantage of a larger value than 1.8 for the parameter factor used by the writer in this instance is that it permits of flattening the curve just down stream from the crest, thus improving the flow conditions. For a solid masonry weir a larger parameter enables the parabolic section to be extended to a lower elevation than otherwise. In the case under consideration it was obvious that the comparatively steep apron slope required a minimum value for this particular parameter.

The author intimates that the preliminary soundings were made by test pits and auger borings, that failure occurred where the up-stream cut-off wall extended only from 5 to 7 ft. into the over-burden, and that the technical advisers consulted on the original project were not familiar with the local conditions. Later, it appears, the results of core borings driven for reconstruction investigations disclosed lentils and alternations of pervious with impervious strata, and that sandstone boulders or false cliffs had been mistaken for bed-rock beneath the original cut-off construction.

These facts cannot emphasize too strongly the great importance of extensive sub-surface investigation preliminary to construction such as this, and the author's words in regard to foundation conditions as uncovered in the subsequent explorations, that they "required the most serious consideration and care", are conclusive.

Mr. [illegible] which may be taken as the actual crest of the weir and the parameter at about 1.5 times the head of full reservoir level on the so-called "theoretical crest" (that is, the actual crest of a corresponding thin-edged weir). This parabola, with axis vertical and through the crest, will determine approximately the lower nappe or sheet of overfall for a weir.

The study leading to this conclusion was based on experiments by M. Bazin. This parameter may be increased for practical considerations to as much as 2½ times the head.

The writer is not aware of the detailed method by which the shape of the new spillway crest of the Stony River Dam was fixed, but, by the foregoing method, he calculated the crest for the given head and was interested to find that, as far as could be ascertained by careful scaling of the outlines of Plate III, a practical identity resulted. The advantage of a larger value than 1.5 for the parameter factor used by the writer in this instance is that it permits of flattening the curve just down stream from the crest, thus improving the flow conditions. For a solid fixed weir a larger parameter enables the parabolic section to be extended to a lower elevation than otherwise. In the case under consideration it was obvious that the comparatively steep apron slope required a minimum value for this particular parameter.

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## AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By Carl Gayler, M. Am. Soc. C. E.

Carl Gayler,† M. Am. Soc. C. E. (by letter).‡—The writer gladly acknowledges the great merits of the work of the Committee, and has no doubt that the report will be for a long time an aid to the concrete engineer as a book of reference, guiding him in his work and correcting or sustaining him in his judgment. Aside from a few objections which can be raised, it will probably be accepted as a standard by the Profession. Still, in the writer's opinion, there are such objections, and he begs to state them. Mr. Gayler.

It is to be regretted that so little prominence is given to the important problem of eliminating the dangers from laitance. Every engineer, active in reinforced concrete work, has been aware of this danger for years, many engineers (among them the writer) have come to its realization only after bitter experience, but few have had a clear conception of the magnitude of the problem before reading the classic paper "Water the Chief Factor in the Making of Good Concrete", by Nathan C. Johnson, Assoc. M. Am. Soc. C. E.

Now, in the Report, Chapter II, Section 2, laitance is not mentioned as one of the causes through which reinforced concrete structures fail, nor is there—and this is more important—any cautioning of the inspectors on this point in Section 3 of the same chapter. The "Destructive Agencies" mentioned are: "Corrosion of Metal Rein-

* Continued from March, 1917, *Proceedings*.

† St. Louis, Mo.

‡ Received by the Secretary, March 16th, 1917.

Mr. Gayler. forcement", "Electrolysis", "Sea Water", "Acids", "Oils", and "Alkalies", but not one word of "Laitance."

The short references in Chapter IV, Section 3, to the removal of laitance can hardly be considered as emphasizing the subject sufficiently, nor is mention made of the chief agency of the trouble, that is, excess of water.

The field covered by reinforced concrete work has grown to such astonishing dimensions that an engineer, no matter what his specialty, to be competent, must have a general knowledge of the principles and requirements of such work. Whether he is engaged in hydraulic works, power stations, bridge or irrigation work, it confronts him in the shape of foundations, conduits, reservoirs, retaining walls, or viaducts. Now, of this immense field, many features of which are still in the midst of evolution, and for which the practising engineer expected enlightenment from the report, not so much on detailed features as through broad, general rules, very little is to be found, with the exception of the indoor slab.

It will be objected that the different types of structures just mentioned do not come within the scope of a report on concrete and reinforced concrete (and, in fact, the Committee makes this claim in the introduction to the report), that such a report is complete after full presentation of the qualities of the materials and of the principles underlying their wise and economical use. Even granting the force of this objection, it is thought that by allowing one-tenth of the space allotted to the flat slab, with its dropped panel, column capital, wall girders, etc., to a few other types of structures of general interest, the value of the report would have been much increased. Some expression of opinion, for instance, on the problems of skeleton retaining walls, on the advantages of the hinged arch, slabs of bridge floors (very different, indeed, with their delicate problems of drainage and, shrinkage, from the indoor slab), etc., in the form of concise résumés of the experience of the eminent engineers to whom we owe this report, would have been of great value to the Profession.

Reinforced concrete structures are, essentially, heavy structures. Their success and appearance depend on unyielding support during construction. In many important cases the designing of falsework and centers requires as careful study as the planning of the superstructure; and their maintenance, until the full completion of the work, as much attention as the depositing of the concrete. A reference in the report to this subject, perhaps with the addition of permissible stresses in the materials used, might have been expected. The case is entirely different from the additional types referred to previously. Falsework and centers may be said to form, during construction, an integral part of the superstructure, and it seems that a general report on concrete and reinforced concrete is not complete if this subject is omitted.

There is one other point on which the writer takes exception to the report, that is, the exclusion of high-grade steel. The Committee has been careful in the wording: it merely recommends the structural grade, but the unit stresses, adopted in the report, practically eliminate the use of high-carbon steel. Mr. Gayler.

High-carbon steel is in general use for reinforced concrete work. It is specified, on even terms, with structural steel, in the standard specifications of the American Society for Testing Materials, of the American Railway Engineering Association, in the building codes of our larger cities, and in the specifications for highway bridge work; and railroad and municipal engineers, as well as engineers in private practice, use it.

As no explanation is given in the report, of the Committee's preference for the milder grade, it may be assumed that the question of greater brittleness of the harder steel has been the deciding one. On this point it is found that Taylor and Thompson, in their treatise on "Concrete, Plain and Reinforced", in a lengthy and clear discussion (p. 414) on the advisability of using hard steel, in which due stress is laid on the difference in the use of steel for bridge work and for reinforced concrete work, and in which careful inspection at the mills is insisted on as an efficient means of obviating dangers from brittleness, end the paragraph with the following (as it seems to the writer) unanswerable argument:

"Steel which can be employed with safety for all locomotive and car wheels of the country certainly cannot be discarded as unsafe for concrete, provided similar precautions are taken in its purchase."

From his own experience, the writer considers high-carbon steel for reinforced concrete work a thoroughly safe material. With proper care and judgment used, where any bending is to be done, no undue risks are run.

Assuming now the safe use of hard steel to be granted (its general use cannot be denied), we reach the important question whether steel with a higher percentage of carbon should be proportioned for the same low unit stress as milder steel (always assuming that the conditions imposed by the bond stress are fulfilled). As far as general practice goes, the answer has been, almost universally, in the negative.

The difference in the unit stresses allowed for high steel over structural steel may be stated, approximately, to be as follows: In railroad work from 2 000 to 3 000 lb.; highway bridge work from 2 000 to 4 000 lb.; and city building laws from 4 000 to 6 000 lb. (the building laws of the City of St. Louis, for instance, specify 14 000 lb. for medium steel and 20 000 lb. for high elastic limit steel).

Considering that, in the report, not less than fifteen different unit stresses are specified for stone or gravel concretes, according to the

Mr. Gayler. nature of the aggregates and the ratio of the mixtures, it seems strange to find the single line "the tensile or compressive stress in steel should not exceed 16 000 lb. per sq. in.", covering the whole subject of proportioning steel, whether the elastic limit is 30 000, 40 000, or 50 000 lb., or the ultimate strength 50 000, 70 000, or 80 000 lb. per sq. in.

Taking into consideration that this question of permissible unit stresses in the steel affects the universal practice in concrete work all over the country, and for every class of such work, there must have been strong reasons for the action of the Committee and engineers hope to find them stated in the concluding answer to the different discussions on the report.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## DISCUSSION ON REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES*

By Joseph Mayer, M. Am. Soc. C. E.

Mr. Mayer.

Joseph Mayer,† M. Am. Soc. C. E. (by letter).‡—For a thorough investigation of valuation and the therefrom inseparable regulation of public utilities, the nature of the economic problem which must be solved and the kind of solution desired must first be defined clearly.

In competitive private industry the owner, generally, is practically at liberty to choose the prices for which he is willing to sell his products, provided free competition is not in any way interfered with.

The public relies on competition to increase automatically the supply and thereby reduce the prices whenever they are such as to produce for a long time much larger average profits in any industry than are secured in other industries offering otherwise equal advantages.

Experience shows that, in many industries producing the necessaries of life, competition is extremely limited or impracticable, and that excessive prices, producing unusually large average profits on investment, thereby arise. Such industries are pre-eminently street railways, the distribution of gas, electricity, and water, telephone and telegraph service, the carrying off of sewage, local, and to a less extent long-distance, railways, and express service.

Capital is obtainable for these industries if the prices of their products or services are regulated so that average profits are the same

* Continued from March, 1917, *Proceedings*.

† Montreal, Que., Canada.

‡ Received by the Secretary, March 22d, 1917.

Mr. Mayer. as in competitive industries. The practical problem is to find such a regulation of monopoly prices as will equalize average monopoly and competitive profits without interfering with efficiency of management.

One method of price regulation is to introduce public ownership and operation, as is done universally with sewers, largely with water supply, and, to some extent, with others of the industries mentioned.

Public operation of monopolies, due to gross inefficiency of management has often proved a failure. Private operation, with public control to secure fair prices without destroying efficiency, therefore, is attempted.

To secure efficiency with private operation, the net revenue of each enterprise must depend on its efficiency of management. This cannot be attained by regulating the prices so that the individual enterprises secure a uniform rate of return on the investment. For fairness, the prices must be regulated so that the average rate of net return on the investment in every regulated industry, not that in individual enterprises, is the same as this average net return in such competitive industries at the same time and place as offer otherwise the same advantages.

The Committee advocates a standard rate of return on the investment in every individual normal enterprise, practically ignoring differences of efficiency. It defines a normal property or enterprise as one which is neither overbuilt, inadequate, nor improperly located. The Committee, though advocating and assuming in its ascertainment of the tangible value a regulation ignoring the existence of differences of efficiency and the consequent fairness of unequal rates of return, endeavors, in a hesitating manner, to correct the therefrom resulting errors in the chapter on "Intangible Values."

The aims pursued by the Committee and the writer become thereby much less divergent. The writer, however, claims that these aims cannot be attained by the circuitous method adopted by the Committee. The Committee tries to follow closely the decisions of the Courts in determining values. The writer believes that, as value is an economic phenomenon, it must be determined by studying economic laws, and not legal decisions merely. Economists, not Courts, are competent valuators. The Courts acknowledge this by only stating correctly what factors influence value and must, therefore, be considered, but refraining from prescribing how value is to be determined from these factors.

The Courts insist on fairness, and keeping promises is the most important guidance in determining what is fair. The writer, therefore, believes that the first task in valuation is to ascertain what promises have been given. This is a political and partly a legal question, wherein the laws and the decisions of the Courts are final. The

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second task is to find what values result from keeping promises, and this is a purely economic question. Court decisions which contradict economic laws cannot stand, and will be reversed as soon as this is clearly demonstrated.

The decisions quoted in part by the Committee on page 1770* indicate that the Courts sustain the determination of value in the ordinary economic manner, that is, from estimated future net revenue and market quotations, and do not sustain the theory that a public service property is entitled to a standard return on either actual cost or cost of reproduction less depreciation, which underlies the assumption that costs are values.

In competitive industry it is held to be fair that the investor obtains a greatly varying rate of return on his investment, the amount of which depends mainly on the industrial capacity of the promoters and managers of the enterprise, but partly on changes in the industrial world occurring independently of the management. Competition is relied on to assure the fairness of prices. The independence of private ownership is only practicable when the owner secures the profit and loss resulting from judicious or injudicious choice of enterprise, from good or bad design, or from other differences in efficiency of management. The Committee proposes in its main argument that the fairness of regulation be judged by the return secured by individual investors. The writer maintains that, with private property, this is not practicable. The return of the individual enterprise must vary greatly with efficiency, and the efficiency can only be practically judged from the rate of return secured under conditions prescribed in some other way than by trying to secure a uniform rate of return in individual enterprises.

The Committee not only advocates such a regulation, for natural practically complete monopolies, as will give a uniform rate of return in individual enterprises, but assumes in much of its reasoning that such, if any, regulation has actually existed in the past and ought to have existed where all regulation was absent. As a matter of fact, no such regulation ever did exist, and never ought to exist. Many of the Committee's inferences in regard to values, depreciation, and the proper future rates based on such assumptions, therefore, are in error. The activity of regulating commissions is generally limited to reducing grossly excessive prices where consumers complain. They then endeavor to fix such prices as will, in their opinion, with average efficiency of management, produce about the same rate of profit as in local competitive industries at the same time. The price adopted is mostly and necessarily, since the required facts are missing, an approximate guess, unsupported by adequate evidence. Actual values of

* *Proceedings*, Am. Soc. C. E., December, 1916.

Mr. Mayer.

enterprises thus regulated, therefore, are widely different from those which would have resulted from the imaginary regulation advocated by the Committee, and which it endeavors to determine.

Where competition remains an important element, no attempt is ever made to secure a uniform return on the investment in individual enterprises, for the very good reason that it would not be practicable. The fundamental principle which governs commerce and should govern regulation and valuation is: "Be careful what you promise and keep your promises."

In granting franchises, in the past, the public, generally, has not been careful in its promises, and has often made it very difficult or impossible, without gross injustice, to keep the promises it made. The economic developments of the present were not foreseen in the past, and the promises made were so extremely indefinite that it is generally impossible to determine accurately what was promised and to keep accurately the promises. In commerce, it is sometimes found just, under exceptional unforeseen circumstances, by a moratorium to relieve merchants of given promises to pay their debts when due.

The case of unreasonable public franchises is in some respects similar, and there are cases when justice requires expropriation on fair terms. Regulation was introduced that was not provided for in many of the franchises, and was frequently modified in such an unexpected manner as to affect seriously the net earnings and thereby the values of enterprises. Though the principle of keeping promises is thus frequently disregarded, it nevertheless remains a valid principle whenever it is possible to follow it without serious injustice. Another necessary principle governing valuations follows from the nature of value.

From the definition of value, cited by the Committee in the Glossary, it follows that the value of a public utility, the securities of which are largely dealt in on the exchanges, can be most closely obtained from a study of the quotations in the stock exchanges, and the amount of securities outstanding. The security quotations represent an estimate by the market of the present value of the future income expected to be paid to the owner of the securities.

This future income and the current quotations measure the value of securities, and they are what the owner values and what the Courts endeavor to protect. The future income depends largely on the future regulation. This latter is not exactly known, but its nature is inferred from the present legally established regulation and the knowledge that the Courts, generally, will not sustain such changes of the existing regulation as will destroy present values, as well as from the necessity of regulating prices so that it will be possible to secure the needed

capital for building new properties, for extending old ones, and for rendering satisfactory service. Mr. Mayer.

To ascertain the present values of future revenue, the rates of interest to be adopted must be known. Such rates must be chosen as will make, for enterprises the securities of which are largely dealt in on the exchanges, the value inferred from future net revenue the same as that inferred from the security quotations. The Committee proposes to find the values of public utilities by determining first the tangible and then also the intangible value, giving the latter due weight.

By strict logic, the total value is equal to the tangible plus the intangible value, and as the intangible value is equal to the exchange or market value, derived from estimated future net revenue and quotations, less the tangible value, their sum, or the total value, is equal to the exchange value.

The determination of the tangible value, therefore, is at best only indirectly useful for the purpose of valuations. The Committee prefers, however, not to be logical. It infers, from the fact that the Courts give round sums for the intangible value, that the entirely indefinite phrase giving due weight to the intangible value must be substituted for the exact request to add the intangible to the tangible value. There is abundant reason for using round figures for the result of any valuation, however determined. It is improper to use exact figures when the degree of accuracy obtainable by any kind of valuation does not justify anything else but round ones. Another reason given is that the Courts do not assert that the value must be found by considering income and quotations alone. It might also be stated that the Courts expressly assert that costs are one element to be considered. The Courts, however, do not state how the value is to be determined from the various elements which must be considered. The cost and the average life of every perishable part of an industrial enterprise must be known in order to determine its annual decretion which is a part of the operating expense. The net earnings, therefore, cannot be determined without knowing the costs of the perishable parts of the plant. For determining what prices must be allowed to secure capital for enterprises capable of rendering the same service, and thereby to choose a fair regulation, the costs of a substitute enterprise are of importance. In a valuation based on net revenue, costs, therefore, are considered in two ways: First, in choosing the regulation to be assumed; and second, in determining the net earnings resulting therefrom.

The Committee endeavors to show that the kind of regulation it advocates does not necessarily destroy efficiency. For this purpose it again departs from logic. It desires a regulation which makes tan-

Mr. Mayer. gible value equal to exchange value. This can only be attained by making the return on the investment proportional to its amount. It then recommends a departure from this uniformity by advocating the London and Massachusetts sliding scale of dividends.

The instance cited is the allowed 7% dividend for 90-cent gas with ⅕% increase of dividend for every cent of reduction in the price of gas. The gas company, under this rule, now obtains 9% dividends with 80-cent gas. In competitive industry, when a producer reduces costs of production he obtains himself the profit from the reduction of cost until the improvement is adopted by enough of the competitors to increase the supply materially; then prices and the profits of all producers are reduced, and the backward ones are compelled to improve, or receive abnormally low profits. After a short time, average profits are the same as before, and the public gets the benefit of the improvement. During the transition period, those in advance gained, those in the rear lost.

With the sliding scale, when an improvement reducing costs is made by any one and widely adopted, all, whether fast or slow, obtain a permanent increase of dividends.

The inducement to make the first trial of improvements is greatly reduced; that to wait until they are well tried out is increased.

Improvements are discouraged, and permanent excessive dividends result from those adopted. The sliding scale is unfair to the public; therefore, it does not spread. The two requirements, fair prices and high efficiency, are not, and cannot be, secured by the methods of regulation advocated by the Committee. They can be secured by improved franchises for all natural monopolies.

The franchises should all be definite competitive operating contracts for limited periods of time.

Such a contract should be based on unit prices for the work performed, which work should be as accurately described and enforced as that described in a contract for building a large engineering structure. The contract must contain such clauses, referring to decretion and the compensation for the plant of the contractor, if any, at the end of the operating period, as will assure adequate maintenance and exact determination of the payments for such plant. If old, the property forming the subject of the operating contract must first be obtained by the public by a valuation based on the present value of the future net revenue with the present regulation, unless good reasons exist for believing that the present regulation is unfair. In this latter case, such a regulation, in harmony with promises made, should be assumed as will make it possible to secure capital for new properties capable of rendering equal service and for extending and satisfactorily

operating old ones. If there is a good market for the securities, the value should also be determined from a study of the quotations. Mr. Mayer.

For valuing the property and, later, for determining the annual decretion, an inventory must be made giving an estimate of the original cost, age, and average life and scrap value of every perishable part of the plant; or, where these are unobtainable, an estimate of the present value, the remaining life, and the scrap value.

For determining every year the decretion of the plant, which is a part of the operating expenses, the decretion must be defined so that it can be accurately determined and so that it agrees as closely as possible with the facts.

The following rules will serve for this purpose:

The nominal annual decretion of any part is the difference between its cost and its estimated scrap value divided by its estimated average life. The actual life and scrap value are different from the estimated average life and the estimated scrap value. To correct the errors resulting therefrom, the following rules become necessary for calculating the actual decretion every year.

For parts in use and younger than their estimated average life, take the nominal annual decretion; for parts in use and older than their estimated average life, take no decretion; for parts discarded during the year, take the cost less the decretion previously allowed for, and consider the actual scrap value as part of the earnings.

These rules take an estimated decretion for parts in use for which the actual one is unknown, and correct the therefrom resulting error as soon as the actual decretion becomes known by the discard and sale of parts. They, therefore, give as close an approximation as practicable of the actual decretion.

By means of the inventory and the rules for decretion received by the bidders, the decreted cost of the plant can be ascertained at the end of every year. If the decretion thus determined is considered a part of the operating expense and placed in a decretion fund to be used for replacements, improvements, and extensions, without thereby increasing the cost accounts, the plant will be efficiently maintained.

New capital is required for most plants, and any operating contract must contain clauses prescribing who is to furnish such capital for extensions and increase of capacity, and the compensation it is to receive. The public is interested in all parts of the plant which may survive the contract, and a board of engineers representing the public must have a controlling voice in selecting and supervising the acquisition of new plant. An operating contract must describe the kind of service wanted as well as the maximum prices to be charged, based, in an old plant, on the previous ones, with such changes as appear advisable at the time of framing the contract. The operating com-

Mr. Mayer. pany should be permitted to reduce, but not to increase, the prices. The contract should be framed by representatives of the public. Invitations to bid should be sent out giving all the available information required by bidders to assist them in calculating the net earnings during the period of operation, which might be about 20 to 30 years. The company should be required to surrender the plant at the expiration of the contract at cost of the parts, if any, furnished by the contractor, with the approval of the representatives of the public, during the period of the operating contract, the decretion fund going with the plant.

The bidders should be asked to give the annual rental which they are willing to pay. The contractor must furnish a guaranty for at least one year's rent, which should be payable monthly out of the net earnings, if possible. If the guaranty fund is encroached on, the contractor should be required to replenish it. A quick foreclosure, in case of non-fulfillment of conditions, should be part of the contract. By such provisions, large contracts can be awarded safely to bidders backed by a moderate capital. This will make real competition possible for every large enterprise.

This, though quite incomplete, will give a general idea of the kind of definite competitive contract that should replace the present indefinite franchises, in which protection of the interests of the public is obtained by commissions having, among many other indefinite powers, that of prescribing and changing without compensation, after award of the franchise, the prices of the products and services furnished by the operating or owning company. Changes are necessary in any long-term contract. Therefore, there must be provision for making them without changing the value of the contract.

The two contracting parties should be on an equal footing, and their respective rights and duties should be defined exactly.

With such a contract, fair prices would be assured by competitive bids for contracts of such size as will assure active competition.

Efficiency is secured because the profit or loss of the contractor depends entirely on his efficiency. The drawing up and enforcing of such a contract does not require the superhuman knowledge and ability that would be necessary for assuring efficiency and fair prices by the present methods of regulation. Only one valuation is required for each old enterprise, that for acquiring ownership of the property. All succeeding transfers would take place for sums based on costs and decretions, defined in advance so that they can be determined accurately.

Reviewing the foregoing discussion: the problem to be solved is to replace adequately the self-regulating competition which has disappeared either partly or altogether in some fields of productive industry.

Where free competition prevails, fair prices and efficiency are secured automatically by increased supply and falling prices, in industries where average profits are excessive, and reduced supply and rising prices where they are abnormally low. The fairness attained is not equal profits for good and bad management but, in the long run, approximately uniform average profits for the same degree of industrial capacity in different industries. Efficiency is secured by profits depending mainly on it. The return on the investment in the individual enterprises of every industry varies widely with the economic skill shown in their promotion and management, and partly with unforseeable economic changes outside and independent of the management. The board of directors, consisting of, or representing, the largest stockholders, has generally no other common primary aim than the, in the long run, largest possible return on the investment. The value of the business may be a small fraction of the investment or may exceed more than ten times its amount. An industrial enterprise is a tool for the production of a future net revenue; its value is the present value of its future net revenue; this, on an average, depends mainly on the cost of production of the business, but, in individual cases, mainly on the industrial capacity of the management. This deviation of the value of such an enterprise from its cost of production is what produces efficiency of promotion and management without public control, and makes private property of such enterprises practicable. Mr. Mayer.

The directors will endeavor to secure, and will retain, the most efficient managers. Were uniform profits guaranteed, as is proposed for the ideal regulation of the Committee of completely monopolized industries, a crowd of incompetent promoters would encumber the field of new enterprises, and the directors would probably choose their sons, nephews, and friends for the leading positions, and would follow the same method in selecting the minor officers.

Since, with uniform profits, the public pays for good or bad management and carries the risks, it will gradually insist on full control of promotion and management, independent private property will disappear, and public operation and ownership will result almost automatically. Uniform profits for individual enterprises are incompatible with private operation, because, inevitably, they destroy efficiency of management. The only possible alternatives, therefore, are public operation, and private operation with profits in individual enterprises mainly depending on efficiency and with average profits in every monopolized industry, the same as the average profits of competitive industries at the same time and place.

The frequent inefficiency of public operation in democratic communities arises from the method of choosing the public equivalent of

Mr. Mayer. the board of directors, and the general manager and his assistants. They are either elected directly or are appointed by representatives elected for other qualities than their knowledge of the industrial enterprises under public management. The voters and their representatives generally know little about these industries.

The management is controlled frequently by the most powerful politicians of the day, and will remain in power through its control of votes. It is often dependent on the politicians in its appointments and discharges of employees and the choice of their salaries and wages, in the prices charged for services rendered to the public, and sometimes, also, in its purchases of materials and supplies as well as in the choice of improvements and extensions.

In attaining and keeping in power, the efficiency of management is a very small factor, and, therefore, is largely neglected.

A private board of directors in a company with a fixed rate of dividends would be worse, but a private board with a rate of dividends depending mostly on efficiency is far superior.

There is, therefore, no reasonable doubt that, as long as the evils described prevail, private operation will secure a cheaper and better public service when the net earnings of the private company depend mainly on efficiency of promotion and management. One way of securing the fairness defined as equal average profits in each monopolized and in competitive industries, combined with the efficiency of management resulting from profits in individual enterprises depending mainly on efficiency, consists in replacing the kind of competition which has disappeared, because it is impracticable, by another kind which is practicable. This consists in acquiring public ownership of all partial and complete monopolies, and selling definite operating contracts for limited periods to the highest bidder. Two necessary features of such contracts, which have been given insufficient attention in past franchises, are an exact definition of the annual decretion and of the price to be paid to the contractor or owner, either when he fails to comply with the terms of the contract or at the end of the operating period.

Another feature of such contracts, not mentioned in the foregoing, is the possibility of prescribing exactly in the contract, without injustice to the contractor, the minimum wages and other working conditions approved by public opinion, and the manner in which they are to be changed without changing the value of the contract.

Near-by economic changes, occurring independently of the contractor and influencing the net earnings, can be foreseen largely by the bidders and allowed for in the bids. Distant changes, which cannot be foreseen, will influence future bids in the same manner. Approximate proportion of efficiency and net earnings will be secured thereby.

In such contracts changes in the value of the lands occupied by the enterprises accrue mostly to the public to whom they belong. They are caused largely by events beyond the control of the contractor. Justice requires, therefore, that they should not affect his revenue. In these respects public ownership with the private operation herein described is more just than private property in competitive enterprises. Another advantage is the much greater stability in the value of all securities representing part of the income of either the property operated or of the operating contract. With publicity of accounts and exact determination of the annual net earnings, the value of the contract can be judged much more accurately than that of a partial or complete monopoly regulated in the present necessarily arbitrary manner. Mr. Mayer.

Transfer of the property from one operating company to the next, based on costs, when agreed on by contract, is far superior and is more just than transfer based on values. Costs can be ascertained accurately when followed from the start, values never; costs make the contracts definite, values indefinite. This change from the general practice of using values instead of exactly defined costs, in defining the terms of transfer, is what makes short-term operating contracts practicable. The other method of securing fair prices, together with efficiency (which are both essential), is to fix, by regulating commissions, from time to time, the kind and quantity of service to be furnished by partial or complete monopolies and the prices which they are permitted to charge.

The prices are fair when they make the average profits in every monopolized industry the same as in competitive ones; they secure efficiency when the profits in every individual enterprise are in proportion to efficiency of management. To fix the prices it is necessary, therefore, to ascertain, first, the average profit secured in competitive industries at different times and places. By profit of an enterprise is meant the percentage of net earnings on its value. The sum of the values of a large number of well-selected competitive enterprises and the sum of their net earnings must be ascertained. The latter multiplied by 100 and divided by the former gives the average competitive profit.

Though the value of individual competitive enterprises varies greatly from their cost of production, the sum of the values of a very large number, representative of all competitive enterprises, is approximately equal to the sum of their costs of production. This results from the tendency of capital to flow to the most profitable investments, which makes the average return of all kinds of otherwise equally attractive competitive investments the same. The value of the sum, but not the value of each, can be determined, therefore, from cost of

Mr. Mayer. production. The average profit of competitive enterprises can be determined, therefore, if the necessary facts are ascertained.

With any given regulation, the average profit in any monopolized industry can be determined in a similar manner from the sum of the net earnings and that of the costs of production of the enterprises. The average profits thus ascertained should be the same in competitive and in every monopolized industry. This assures fairness of the regulation as a whole. If the profits are found to be different, the regulation should be changed so as to make them alike.

The regulation in detail must prescribe different prices in different places with different costs of production. The differences in cost of production in different places with ordinary methods of production, therefore, must be ascertained.

Although it is theoretically possible to regulate fairly in this manner the prices of monopolized industries without destroying efficiency, the practical difficulties are too great for the exact application of this theory. The profit of competitive industries varies from time to time and from place to place, and that of the various monopolies should vary in the same way. The necessary facts are unknown and cannot be determined accurately, and the necessary calculations would be extremely complicated. An enormous staff of regulators would be required to apply the theory.

The various regulating commissions cannot use this method, because they have neither the needed facts nor the staffs to gather them. Therefore, to simplify the method, a tendency toward uniform profits, where the management is not grossly inefficient, is inevitable. Regulation is generally only undertaken where there is complaint of excessive prices, so that the actual regulation is generally extremely sporadic and intermittent, and it does not follow consistently any theory, because this is practically impossible. The decisions of the regulating commissions, therefore, cannot be foreseen, and often appear to be quite arbitrary.

A very serious consequence of the, at present, extremely indefinite relations between the natural monopolies and the public is the uncertainty regarding the prices which will be permitted by the regulating commissions and the consequent large fluctuations in the market values of their securities. These commissions are directly or indirectly selected by, and are responsible to, the public, one of the parties to the implied contract between it and the utility companies. The principal protection of these companies is the impossibility of securing capital for new properties, extensions of old ones, and adequate service, unless sufficient return is allowed on the existing investment. The uncertainty of the return and the fear that it may not be adequate prevent new investment, unless a large return is allowed to compensate for the

apparent risk. The regulating commissions may fix such prices and consequent returns as will attract capital and make it thereby possible to supply adequate public service. Such rates of return may be thought to be excessive by the public, unless the inadequate service with lower ones is felt to be the greater evil. Inadequate service, therefore, may lead to higher rates of return than adequate service, a condition which is evidently not conducive to the best service. Long-continued, inadequate service by private companies will create a demand for a radical change. Unless a way out, which will be an improvement over present conditions, is made widely known and adopted, a successful agitation for public ownership and operation seems to be inevitable. With definite competitive contracts any practicable and desirable kind of service can be obtained by the public by paying the necessary competitive price. As most of the risks incident to arbitrary regulation disappear with definite contracts, investments in them are comparatively safe, and will attract capital at low rates of profit. Most of the capital required would be secured by Government bonds at very low rates of interest. Mr. Mayer.

The law ordering the valuation of interstate railways by a Federal commission was the result of a widespread belief that the capitalization of these roads was largely fictitious and far beyond their cost of production. It ordered, therefore, the determination of the actual costs of production to date and of reproduction less depreciation, so that they might be compared with the known capitalization, in order to ascertain the difference between them.

The Committee has done an enormous amount of valuable work to ascertain the principles for determining these costs and the depreciation. It has not determined values, however, or the correct principles of valuation. What it calls tangible value is not a value, but a cost of reproduction less depreciation. In the writer's opinion, the Committee has not altogether abandoned widespread erroneous views in regard to the nature of value, though it gives, under value and market value, correct definitions of value, of price, and of market price.

The identification of value and price is harmless, as long as the value of money remains substantially constant. The corrections necessary with changes in the values of money are a separate problem, not discussed by the Committee.

The determination of costs meets the demands of the Federal law prescribing it. The unsatisfactory chapter on "Intangible Values" attempts to meet the demands of the Courts which insist on the consideration of revenue and market values. With a slight correction, prescribing the addition of what the Committee calls tangible value, and of intangible value to obtain the total value, its valuation is

Mr. Mayer. correct. The demands made by the nature of economic phenomena, which prescribe the highest laws to which all others must accommodate themselves, however, are but imperfectly met by the investigation of the Committee. The value of an industrial enterprise of any kind is the present value of its future earnings. Security quotations, where available, give the fluctuating opinion of the market on the value of the securities. There are often no reliable quotations; the estimated future earnings with a fair regulation furnish then the means of determining values.

For determining what is a fair regulation, costs and depreciations must be ascertained. The Committee has given the principles which must be followed in determining these, and has thereby done part of the work necessary for a correct valuation, but it has not given the principles which must be followed in a real valuation based on revenue and quotations. The task which is now being performed by the Federal Commission for Valuation is, in reality, a cost determination and not a valuation, which latter will not be attempted. The Commission thereby conforms with the spirit and real purpose of the Federal law. Its work will be very useful in improving present methods of regulation of public utilities, but will be inadequate for valuing them.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## UNUSUAL COFFER-DAM FOR 1 000-FOOT PIER, NEW YORK CITY

### Discussion.*

By Messrs. Frederic R. Harris, D. A. Watt, C. A. Wentworth, Thomas H. Wiggin, T. Kennard Thomson, Charles S. Boardman, and William M. Black.

Frederic R. Harris,† M. Am. Soc. C. E. (by letter).‡—The author has presented his subject so clearly and fully that extended comment is unnecessary. Mr. Harris.

On first reviewing the paper one is impressed particularly with the large sum of money involved for the construction of only 220 ft. of pier and the excavation of areas adjacent to the pier sufficient for three slips. However, when it is considered that it was necessary to limit the projection of the pier outshore, it will be realized that the sum expended for the inshore construction was not exorbitant. It must also be taken into account that the greater part of the steel sheet-piling was salvaged and was available, therefore, for use on other work.

In coffer-dam construction, as in other constructions of this class, the unknown element confronts the engineer. Officers of the Corps of Civil Engineers of the Navy have been forcibly impressed with the hazards accompanying construction of this class, as there is little room for doubt that most of the delays and failures in naval dry dock construction are traceable to weak or faulty coffer-dam construction. When both the design and construction of a coffer-dam are

* This discussion (of the paper by Charles W. Staniford, M. Am. Soc. C. E., published in February, 1917, *Proceedings*, and presented at the meeting of March 7th, 1917), is published in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Washington, D. C.

‡ Received by the Secretary, March 3d, 1917.

Mr. Harris. imposed entirely on the contractor, there is a strong temptation to save on first cost and perhaps to take chances on additional and unnecessary risk. The contractor is seldom to blame for this condition, as he is often forced into an untenable position by the double competition involved, first, in competitive bids in which his estimated cost of the coffer-dam which he plans is a part; and, second, in which the saving that he may believe he can make on the coffer-dam determines his profit, or, worse still, a decrease in his loss.

The writer has been much interested in the construction work described, having discussed it quite frequently with Mr. Staniford, Messrs. Holbrook and Bryson, of the Holbrook, Cabot and Rollins Company, and the Commissioner of Docks and Ferries, Mr. R. A. C. Smith, and has been influenced largely by Mr. Staniford's able and conservative engineering plans, as shown by this work, to depart from the precedent heretofore established in the plans and specifications for naval dry docks. This departure, although it has not taken the exact form of designing the coffer-dam required for the work, has, nevertheless, laid down and specified a minimum coffer-dam requirement that would be acceptable to this extent, attempting, as Mr. Staniford did in the work described, to minimize the risk and safeguard against the probability of failure, with its attending losses of money and time.

The fact that the coffer-dam described by the author held and served most satisfactorily the purpose for which it was constructed, is evidence of the soundness of the judgment used in its design.

The writer is particularly interested in the tests made to secure information on the behavior of the rip-rap embankment under various conditions of pressure. The following are the results of computations made from the data in the paper:

In Test No. 1, assuming the head to be 3 ft. 6 in., the active pressure was found to be 360 lb.; the pressure at impending motion, 1 455 lb.; and actual sliding on the bottom 2 800 lb., assuming the same coefficient of friction on the bottom as that of the material itself, and no friction on the board to which the force is applied. These assumptions may account for the figures given being slightly larger than those obtained in the test.

In Test No. 2, the head assumed was 2 ft. 9 in. The computed active pressure found was 250 lb.; the computed impending motion, 970 lb.; and for sliding, 1 460 lb., assuming a plane of rupture through the toe and no friction on the board to which the force is applied.

In Test No. 5, the head was assumed to be $1\frac{1}{2}$ times the distance of the application of the force from the top. At impending motion, the computed force was found to be 500 lb. and the sliding on the plane through toe to be 680 lb.

From observations of the curves shown in Fig. 13, it is believed that the initial movement was probably purely local, and that the impending motion began at the initial points of the curves shown. The agreement of the computations with the critical points of the curves is remarkable and partly verifies such an assumption. Mr. Harris.

D. A. WATT,* M. AM. SOC. C. E. (by letter).†—It would be of interest to know whether any piles were found, during the removal of the pockets, which had been driven out of their interlock with the next piles. In the writer's experience, this is by no means uncommon where the driving is hard, for instance, with closing piles, although no indication of the break may be apparent at the time. Thus, when removing the coffer-dam for the *Maine* at Havana, more than one case was found where one pile at one-quarter or one-third of its length from the end had broken away from its neighbor. Fortunately, the breaks were all below the level of the harbor bottom, which doubtless was the reason the cylinders had not burst, as the surrounding material (mud and clay) prevented the embedded piling from spreading. The piles had been driven from 30 to 35 ft. into this material. During the construction of the coffer-dam one of the cylinders broke open, due to the closing pile being driven out of the interlock, but in this case the weak point appeared to be above the harbor bottom, where there was no outside material to counteract the strain. Mr. Watt.

Similar conditions were found in the later steel-pile-pocket coffer-dam at Troy, N. Y., where the piles were driven into river gravel. The penetration was not more than 8 or 9 ft., as the gravel was compact; yet, when removing the piling, several cases were found where the interlock had driven out. As with the *Maine* coffer-dam, the breaks appeared to be below the surface of the bottom, and no harm resulted.

The saturation of the filling in coffer-dams of this type, referred to by the author as one of the sources of internal pressures, appears to exist at a higher level than is usually assumed, although varying naturally with the porosity of the filling. The filling used for the *Maine* cylinders varied from a sandy to a heavy clay, and in all cases the saturation line was very high. It began at about mean tide level on the outer faces and sloped downward at about 5 horizontal to 1 vertical. The material, therefore, was dry only to an average of about 5 ft. below mean tide; the remainder stayed in a fluid condition. Efforts to lower the saturation level and to drain the filling with long perforated pipes were useless; the pipes would fill up in a few hours after being pumped out, and even near the surface these did not seem to affect visibly the adjacent seepage. This high and constant line of

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* Albany, N. Y.

† Received by the Secretary, March 9th, 1917.

Mr. Watt. saturation was noteworthy because the total seepage into the coffer-dam was extremely small (about ½ cu. ft. per sec.), and there was as much opportunity for the water to leak out of the cylinders into the inclosure as to leak into them from the bay. In the Troy coffer-dam, with the pockets filled with sand and gravel, the water line was naturally much steeper. Judging from the height of the seepage appearing on the inside faces, it was about 2 horizontal to 1 vertical.

The writer visited the 46th Street coffer-dam during its construction, and again after it had been in use for some months. Mr. Staniford and the others responsible for the work are to be congratulated on their success in handling a very difficult problem.

Mr. Wentworth. C. A. WENTWORTH,* M. AM. SOC. C. E.—The Society has received a valuable contribution in this paper, describing an advanced step in coffer-dam construction to meet peculiar and difficult conditions. The tests made in preparing for this work show a foresight which is too often omitted. A comparatively small expenditure for borings and tests at the outset of a difficult piece of foundation work frequently saves much larger expenditures later. The entire responsibility of determining the method to be used in carrying out a given design is too often left entirely to the contractor, instead of considering the method and final structure desired as parts of the same construction, both of which are essential to a completed whole.

The relative amount of yielding in the rip-rap embankment of the large coffer-dam confirms the results of the small-scale tests remarkably well, and the description in the paper gives all the points necessary for use in applying this method to similar work. The difficulties encountered in the construction were surprisingly small, considering the magnitude of the work; and there is evidence of careful preparation in laying out and carrying on the work which is a credit to both the engineers and the contractor.

A coffer-dam of the open type, without cross-bracing, must fulfill two functions: First, it must provide a cut-off wall sufficiently tight to hold back the water and prevent it from opening channels, through or under the dam, which might endanger its stability or cause a seepage flow which could not be cared for by the pumps with reasonable expense. This does not mean that an absolutely tight cut-off is necessary, and, in fact, a water-tight cut-off is seldom obtained. It does mean, however, that large leaks or seepage must be stopped.

The second function which a dam must fulfill is to provide sufficient stability to resist the total pressure of the water. This stability depends on two factors: The total mass, and the internal cohesion of the mass, or its resistance to deformation. In the coffer-dam under discussion, the cut-off consisted of a double row of steel sheet-piles,

* New York City.

interlocked and driven to rock, and sealed with a clayey mud. This cut-off was ideal as a stop for percolating water. The rip-rap furnished —in the most compact form of any loose material—the necessary mass, with the maximum stability attainable, short of a coherent structure. Mr. Wentworth.

In this case there was a coffer-dam requiring a cut-off wall 68 ft. in height. Engineers must expect to encounter these problems more frequently in the future, and the methods used must meet the conditions. The methods applicable in any given case should be governed by local conditions and the cost of available material. When this work was started, 2 years ago, the cost of steel piling was low, and contractors were seeking places for the cheap disposal of rip-rap from the subway excavations. These two factors undoubtedly had much to do with the type of coffer-dam selected. A third factor which defined the limits of the work was the rock bottom, which made an unyielding support, both for the steel cut-off wall and the rock fill. A variation of any one of these three factors, such as a high cost for the first two or a different character of bottom, might have made some other type advisable. Each of these factors should be given its true weight in considering the use of similar construction in other work.

If the same work were to be done again, with the present high price of steel, and rip-rap being more expensive, some other means might well be considered, such as the use of open concrete caissons sunk by the dredging method and weighted by filling with mud, or timber caissons similarly sunk and weighted. These two types would represent coherent structures in which the minimum quantity of material is required to resist the pressure, and their use in this or any other case would be entirely a question of relative costs.

Another dam which should be considered is the ordinary timber sheet-pile and earth-fill type. Such a dam would necessarily have to be placed a greater distance from the bottom of the excavation in order to provide room for the greater mass and flatter slopes required, and also to provide a longer distance for the travel of seepage which might pass a cut-off wall not carried to rock. The latter method would not meet the restricted conditions at the north and south sides of the coffer-dam under discussion, and, in fact, the general method of construction was varied at these points to suit local conditions, large circular caissons being used at the southwest corner, and a single line of sheeting along part of the north side.

These alternates are pointed out in order to call attention to methods which might well be considered in connection with a similar problem under different local conditions.

The paper suggests some thoughts which may not be out of place in this discussion. Harbor development in the United States is now at a point where new methods and types of construction will be required

Mr. Wentworth. to meet the increased draft of ocean vessels, and to provide for a greater permanence in the design of docks and bulkheads than has prevailed in the past. Timber has been cheap and of excellent quality, and has served its purpose well, for coffer-dam and dock work, for depths of water up to about 30 ft.

Most of the docks in the United States are built of timber, usually untreated. The life of work of this class is short, unless entirely submerged in waters free from marine worms, but it has had the advantage of cheapness and rapid construction, and these have been the deciding factors in a rapidly growing and changing community. In the teredo-infested waters of southern and Pacific ports, creosoted timber and other forms of protection have been a necessity for wooden piles, and, even with such protection, the increased cost and decreased life of dock structures built of this material have made the need of more permanent materials desirable.

For permanent docks in depths of 40 ft. or more, which are now required for the largest ocean vessels, timber is getting beyond its reasonable limit, and the use of concrete or masonry walls or piers, or supporting dock structures by concrete caissons, will probably be the next step forward to meet these new conditions of permanency and depth. These methods are suitable, not only in waters where the teredo is not found, but also for most conditions encountered where a permanent dock is required.

Concrete piles have been used in many places infested by the teredo, but they cannot be considered as permanent in waters subjected to ice and frost action, and their permanency in tropical waters is questionable, due to the greater chemical activity of the dissolved alkaline salts in the warm water and the great rapidity with which the steel reinforcement disintegrates when the necessarily thin protecting layer of concrete is broken or punctured.

No engineering work is immutable, and permanency, when applied to harbor work, can only be considered as relative; but it is evident that a new era has been reached in constructing works of this kind, especially in ports like New York. It is a pleasure, therefore, to see the substantial design of the dock walls constructed inside the coffer-dam for the Forty-sixth Street Pier, and it is hoped that this work may be a precedent for future construction.

Mr. Wiggin. THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The author very modestly calls this an unusual coffer-dam. A stronger term might very properly have been used, as will be agreed to, the speaker thinks, by any one who has had experience with such work. The construction itself, after the coffer-dam was completed, and even the methods of building the coffer-dam, were not in themselves so bold, because, though requiring

* New York City.

great skill and ingenuity on account of unusual dimensions, they involved methods of pile-driving and excavation which are sufficiently well standardized; but the conception of backing so high and so large a coffer-dam with a mass of rip-rap of comparatively small thickness was certainly one which required boldness, care, study, and sureness, in order to warrant a construction, in that limited space, which did not give opportunity for increasing the strength of the backing, in case it should be found insufficient. Mr. Wiggin.

It seems to the speaker that the experiments undertaken to discover the action such a structure might take, deserve more emphasis than the author gives them. They were, apparently, the only quantitative proof of the design. The speaker made a few rough computations, and would be interested if the author would include an outline of such computations as were made by him to justify the proportions of the design. For example, the prism of rock which supported the steel walls on the side next the river, appears to have had a weight of about 200 000 lb. per lin. ft., and the water pressure against it appears to have been approximately 100 000 lb. This would indicate the necessity for a coefficient of friction of about 50% in order that the rock fill should withstand successfully the inward pressure when the coffer-dam was unwatered. The experiments appear to show that the coefficient of friction at the ultimate strength of such a "granular" structure, when subject to sliding pressure, would be considerably greater than 50%, perhaps half again as large.

The margin, then, was apparently safe, but not large, and the considerable movement of the structure indicated that just about the right degree of conservatism was used in the proportioning. If the structure had been a little lighter, it might have exhibited signs of distress as the water was lowered inside, and, as there was little more room for additional rip-rap inside, a determination of the next step would have been a serious problem. No such distress occurred, however, and the design was justified, which must have been a source of great gratification to the author and his assistants, and is reason for admiration by the Engineering Profession.

The paper leaves little to be added, though its descriptions are given with a minimum of words. There are a few points, however, which might be explained in more detail. The first is the use of wash-borings and steel-shod piles for the purpose of finding the position of bed-rock. Under many circumstances, neither of those methods would have been reliable. The speaker knows of areas where rock has been located in great detail by wash-borings, but, on excavating, no rock was found; and, similarly, with the driving of rods or piles, it is difficult to know that one does not strike boulders. The speaker presumes that the Dock Department was familiar with the general nature of the shore, and felt confidence in the methods; otherwise,

Mr. Wiggin. that a few borings of another sort, that is, diamond drill or core borings, would have been made in order to be sure that certain critical points were well established.

It is noted that along the north side of the coffer-dam a single row of steel sheet-piling was used. Of course, two rows give more certainty of cut-off than one, but it would be interesting to have Mr. Staniford explain whether a single row would not have done elsewhere. The principal stability appears to have been derived from the rock fill inside the steel piling, and it would seem to be possible to use a single row and back it up so well with rip-rap as to make the cellular construction unnecessary. The speaker visited the work several times during construction, and noted particularly that the cells of the steel piles along the south line were very much distorted by the pressure. They were no longer cells, for some of them were convexed toward the interior of the coffer-dam on both sides; that is, they had simply been forced over by the action of the outside fill so that they constituted not a row of cells, but two rows of piling with certain cross-diaphragms. Considering the amount at stake, the speaker is not at all surprised that two rows, or even more, should have been thought desirable as a factor of safety; but, after the experience, the point of view as to whether one row would be sufficient for the future would be of interest.

It is noted that the steel piling manufacturers have established a strength of about 9 500 lb. per lin. in. for the interlock. This is an important unit to remember in connection with such steel piling, because it has been well established by tests, all giving substantially the same figure, and it will be possible, in many cases, to use such piling locked together as a sort of suspension for supporting pressures.

There are a few places in the paper where the derivation of certain formulas, or, at least, a reference to their derivation, would add to the completeness; for example, on page 122*, there is a factor of 57% which presumably comes from the Rankine sine formula for active pressure. The formula on page 129*, for coefficient of friction, would be clearer for some explanation.

In one or two places, also, references to slopes are ambiguous, such as a slope of ½ to 1, or 3 to 1. The wording leaves doubt as to whether the horizontal is the larger or the lesser figure in the ratio.

In connection with this paper, mention might be made of some steel piling work which the Board of Water Supply did on Staten Island. The Narrows siphon is a 36-in., flexible-jointed, cast-iron pipe, laid across the Narrows, from Brooklyn to Staten Island. On the Staten Island shore, it had to be laid below the level of the bottom of a future dock, and as the material for that dock had not been exca-

* *Proceedings*, Am. Soc. C. E., February, 1917.

vated, it was necessary to carry a trench for a length of about 250 ft. through a firm sandy material varying in depth from about 10 ft. at the toe up to about 50 ft. near the shore, the maximum depth obtaining for some distance. This pipe was laid under water, so that it was not necessary to unwater the excavation, which, therefore, was not a coffer-dam, but merely a sheeted trench with earth and water outside and water alone inside. Mr. Wiggin.

The first design considered was the use of interlocking steel piles, laid in straight lines, with wooden rangers and cross-bracing. That, undoubtedly, would have been successful, but it would have required the placing by a diver of a good many pieces of timber for the rangers, posts, and braces.

It was finally proposed to sheet the trench with interlocking steel piles driven in arcs festooned between steel master-piles. The sheet-piles carried the load as a suspension in a horizontal plane between the master-piles, which were held apart by wooden braces. This design differed from that first described in substituting suspension systems for wooden rangers, an arrangement which incidentally eliminated the bearing across grain of braces on rangers, which is a source of weakness.

The sheet-piles were supplied by the U. S. Steel Company, and were $12\frac{1}{2}$-in. and 38 lb. per yd. The so-called master-piles were spaced about 15 ft. apart along the trench; each consisted of a 12-in., 50-lb., I-beam, with a $12\frac{1}{2}$-in. sheet-pile riveted to one flange and a 12-in., 25-lb. channel to the other, to give it additional strength.

Many of these sheet- and master-piles were about 65 ft. long. The arcs of festoons were 15 ft. in radius, which, under the assumptions of earth pressure, caused a theoretical tension in these curves of about 3 000 lb. per lin. in., as compared with 9 500 lb., which was the supposed strength of the interlock.

There was difficulty, as was expected, in driving the master-piles exactly plumb and in correct position through the firm material. After they were down, each one was carefully surveyed by locating two points on it at the top as far apart vertically as practicable, and then projecting the line of these points to the position at the bottom. An investigation was then made as to what curvature would really result, making due allowance for the bending of the piles and inaccuracies in the location measurements, and wherever the curvature to be expected was less than about one-half that which was tried for originally, another sheet-pile was put in the arc, thus increasing the middle ordinate of the curve and reducing the stress.

T. Kennard Thomson,* M. Am. Soc. C. E.—Mr. Staniford deserves the thanks and congratulations of the Engineering Profession for the marvelously successful accomplishment of a most difficult engineering Mr. Thomson.

* New York City.

Mr. Thomson. problem. Commissioner Smith also deserves the thanks of every one in New York City and State, as well as of many outside these limits, for his courage, ingenuity, and success in securing for Manhattan the 1 000-ft. docks for which there is such urgent demand.

The author's experiments are very interesting; but, as a rule, it is almost impossible to make such experiments really conclusive, on account of the great difficulty of allowing for all the conditions of Nature. For instance, in a small model, a pile representing a rock fill dumped into place would probably have a considerably smaller percentage of voids than in the actual work, and some of these voids might occur in such a way that the subsequent breaking of a few stones might cause a sudden slide, with injurious results, especially if the stones are coated with slime, which condition, also, could not be obtained in the model. Then, again, the formation of ice might be disastrous, although difficult to make allowance for in the experiment.

A coffer-dam or retaining wall, supporting a fill, built of such proportions as to be absolutely safe under most conditions, will fail completely under a load of semi-liquid material of the same weight and height, due to the fact that it will act as a sort of ram. It is always very difficult to estimate the effect of water on a fill, during or after construction, as the cumulative effect is much greater in some materials than the hydraulic head would indicate.

In sub-surface work or excavations, it is even much more difficult to estimate the real pressures on a coffer-dam or retaining wall. The materials have been placed, for the most part, by Nature under great pressures; and are often cemented together by clay or other natural cements, and by roots, stones, etc., so that considerable excavation can often be made without any coffer-dam or bracing. These conditions cannot be reproduced in ordinary experiments, and, in actual work, are often suddenly reversed by the action of springs, rains, etc.

Any experiments, therefore, which are intended to reduce the sections of retaining walls, dams, or coffer-dams below the standards adopted as the result of many experiments, theories, and actual examples, should be checked with the utmost care and foresight.

The author refers to the use of manure for stopping leaks. A very simple expedient, often used successfully in coffer-dam work, where the leaks are small but persistent and troublesome, is to drop an occasional shovelful of ashes or cinders in the water above the leak. This would probably also remedy many leaks in concrete and masonry dams and thus prevent much serious damage from ice forming in the cracks and crevices.

Although too much credit cannot be given to Commissioner Smith and Mr. Staniford for conceiving and constructing this greatly needed

improvement, one cannot help comparing it with a surgical operation, for instance, for appendicitis. Both are expensive, and both are absolutely necessary. In this case it was necessary to remove 12 acres of valuable New York City real estate to make place for water. Mr. Thomson.

The number of times such a surgical operation as the removal of the appendix (or city real estate) can occur is limited, of course, as far as the individual man or place is concerned, yet Manhattan needs many more 1 000-ft. docks—in fact, all that can be obtained—but the upper Hudson is not the most accessible place for them.

Six years ago, the speaker was much impressed with the wisdom of the War Department in refusing to allow further encroachment in the Hudson River, in spite of the very urgent need for such docks; he has always felt that there is a correct solution for all problems, if one only has the patience and ingenuity to find it.

In this case, when extension to the west was forbidden, and to the north and east was impossible, the only direction left was the south, it occurred to him that an extension might be made from the Battery to Governor's Island. Such a course, however, would obviously only accentuate a bad condition, for the City is too long and too narrow as it is. Later, the idea occurred to him that the City might be extended from the Battery 4 miles down the bay, with a width of about ¾ mile, and that a series of tunnels to Staten Island might then be constructed. The City Hall would then be in the center of the city, instead of at its southern end.

The next step would be to form a new bay between Staten Island and Sandy Hook, and introduce many other improvements, with which most engineers are familiar. The net result of this plan would be 50 sq. miles of new real estate and 100 miles of additional water-front. Some 6 or 7 sq. miles could be used for a free port between Staten Island and Sandy Hook. This, alone, an eminent banker has declared, would prove to be the salvation of New York.

Hamburg had to evict 16 000 people, as well as place drastic restrictions on many miles of river front; and even Bremen and Copenhagen were much restricted in their efforts to secure enough water and land space for their invaluable free ports. New York, however, can obtain all the land required without interfering with any one's interests.

Charles S. Boardman,* M. Am. Soc. C. E.—The speaker wishes to call especial attention to the design of this coffer-dam. Mr. Staniford claims that a precedent for the design is found in the coffer-dams built by the United States Government at Black Rock Harbor, Buffalo, N. Y., and for removing the wreck of the battleship *Maine*, at Havana, Cuba. This is entirely true as to many details, such as the use of long steel sheet-piling, especially in the *Maine* coffer-dam, for the mate- Mr. Boardman.

* Buffalo, N. Y.

Mr. Boardman.

rial in which each was constructed was harbor silt below about 35 ft. of salt water. The foundation of the *Maine* coffer-dam was in clay, but the steel sheet-piling for the Forty-sixth Street Pier was driven to bed-rock. It is true, also, in comparing the Black Rock coffer-dam, when it is considered that in that structure the steel sheet-piling was driven to bed-rock, although through good sand, clay, and gravel. Both the Black Rock and *Maine* coffer-dams differ in so many other details that neither can be considered a precedent for the design of the coffer-dam for the Forty-sixth Street Pier, New York City.

Steel sheet-piling has been used in cellular-form, gravity-type coffer-dams, at Black Rock, Havana, Cape Fear River (Browns Landing), N. C., and Troy, N. Y., but never before as a cellular-type cut-off wall in a gravity-type earth and stone coffer-dam.

The form of the cell or pocket used by Mr. Staniford—with straight partition walls and curved longitudinal walls—has been developed from experience obtained at Black Rock and at Havana. It may be of interest, therefore, to follow this thought more in detail, giving a brief history of these two coffer-dams.

Mr. Staniford states:

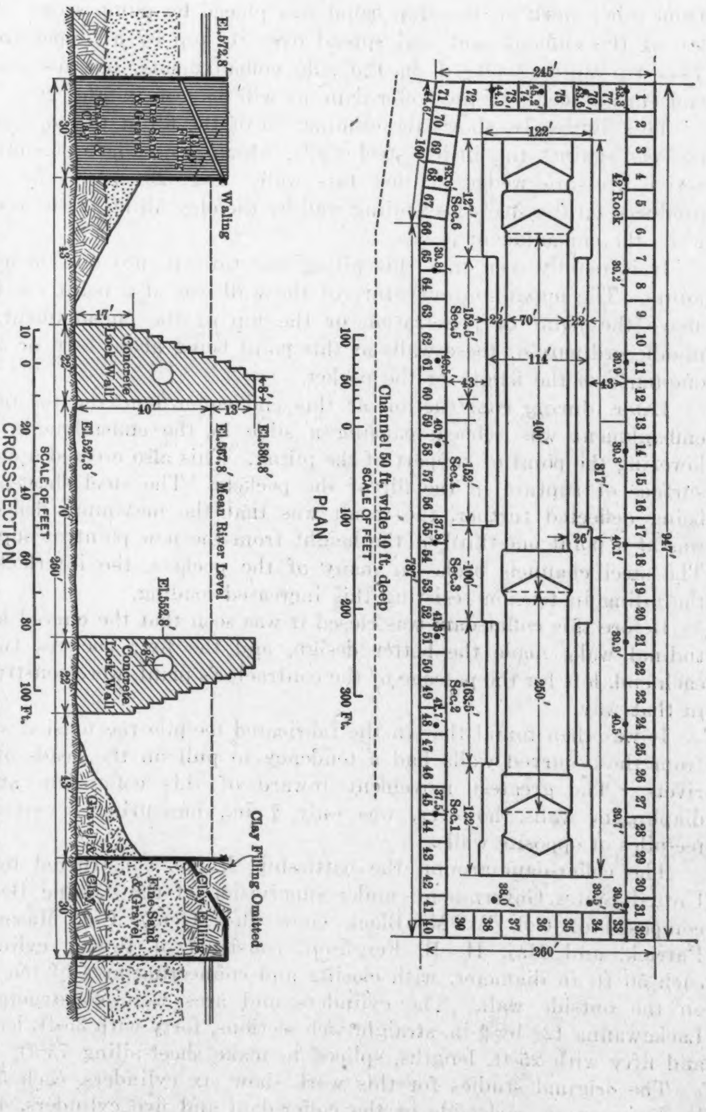
"It was expected that the filling within the pockets [at Black Rock and Havana] would make them stable against the external pressures, unaided, but in both cases it was found necessary to place an embankment of stone against the pockets, and in the case of Havana, extensive bracing to the *Maine* was also placed, in order to stop the continuous inward movement of the coffer-dam."

This statement is correct, but the stone embankment was placed in each case for an entirely different reason than in the Forty-sixth Street Pier, where it forms part of the original design of the coffer-dam. This detail will also be referred to in a brief history of these coffer-dams.

The Black Rock coffer-dam (Fig. 26) was constructed in 1908 and 1909, and consisted of seventy-seven pockets, each 30 ft. square, formed of Lackawanna steel sheet-piling, in lengths ranging from 41 to 54 ft. and driven in straight walls. All the piling was driven to bed-rock, making an average of 33 ft. of penetration through hard materials. The maximum penetration was about 45 ft., at the extreme west wall. Fig. 27 is a view, looking south, showing Pocket No. 40, the northeast corner in the foreground. In the background may be seen the City of Buffalo, the Niagara River, and the International Bridge. This photograph gives a general idea of the location of the ship lock in relation to the river.

The excavation was done by a dipper-dredge having an arm capable of digging to about Elevation — 45 ft., leaving from 3 to 6 ft. of clay and sand over the bed-rock.

Mr. Boardman.




PLAN AND CROSS-SECTION OF BLACK ROCK COFFER-DAM AND LOCK

FIG. 26.

Mr. Boardman.

The slopes of the materials forming the embankment by this dredging method were not maintained as prescribed in the Government engineer's plans, and, to re-establish these slopes, rock obtained from other work on the ship canal was placed by dump scows at the toe of the embankment and spread over its top by a dipper-dredge. This rip-rap was placed on the side embankments, but not on the end embankments of the coffer-dam, as will be seen in Fig. 28.

This figure also shows the elimination of the fill at the tops of the pockets against the inside steel walls, which would have formed an active pressure wedge against this wall. Fig. 29 shows the effect produced on the steel sheet-piling wall by the clay filling in the pockets when the embankment is low.

It is readily seen that this piling was thrown into tension at the joints. The maximum curvature of the wall was at a point one-third above the point of penetration or the top of the embankment, the middle ordinate of these walls at this point being about 3 ft. or about one-tenth of the length of the pocket.

Later, during construction at this end coffer-dam, the toe of the embankment was robbed, causing a slide in the embankment, thus lowering the point of support of the piling. This also created a curved surface of rupture in the fill of the pockets. The steel sheet-piling being deflected further, the result was that the maximum deflection was at a point one-third of the height from the new point of support. The steel channels broke in many of the pockets, the interlocks of the piling in tension resisting this increased loading.

Before this coffer-dam was closed it was seen that the curved longitudinal walls made the better design, and the four pockets, two at each end, left for the passage of the contractor's plant, were constructed in that way.

It was then found that in the fabricated tee-pile the tension strain from these curved walls had a tendency to pull on the heads of the rivets. The greatest movement inward of this coffer-dam at the diaphragm walls, however, was only $\frac{3}{4}$ in., measuring between the tee-piles of opposite walls.

The coffer-dam around the battleship *Maine,* constructed by the United States Government, under supervision of the "Maine Board", composed of Col. W. M. Black (now Brig.-Gen.), Col. Mason M. Patrick, and Maj. H. B. Ferguson, consisted of twenty cylinders, each 50 ft. in diameter, with closing and connecting arcs of ten piles on the outside wall. The cylinders and arcs were constructed of Lackawanna $12\frac{3}{4}$ by $\frac{3}{8}$-in. straight-web sections, forty with 35-ft. lengths and fifty with 25-ft. lengths, spliced to make sheet-piling 75 ft. long.

The original studies for this work show six cylinders, each 50 ft. in diameter on each side of the coffer-dam and five cylinders, 40 ft.

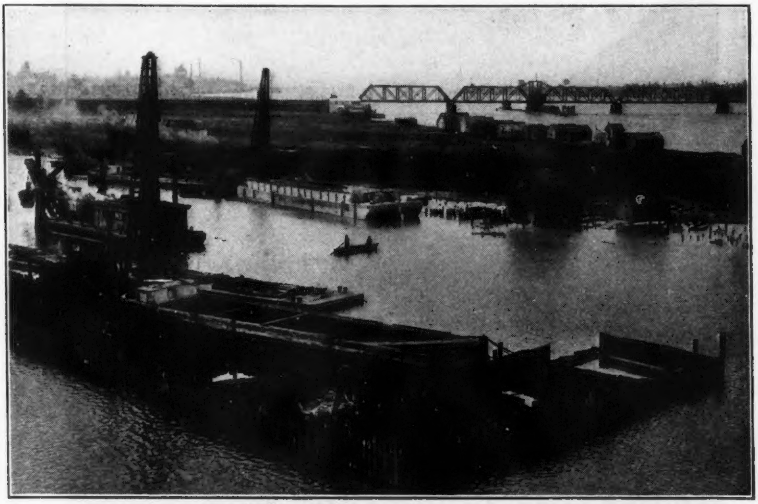


FIG. 27.—BLACK ROCK COFFER-DAM, LOOKING SOUTH. POCKET NO. 40, THE NORTHWEST CORNER, IN THE FOREGROUND.

FIG. 28.—BLACK ROCK COFFER-DAM, LOOKING NORTH FROM SOUTHWEST CORNER. WATER 29 FT. BELOW NORMAL LEVEL OF NIAGARA RIVER.

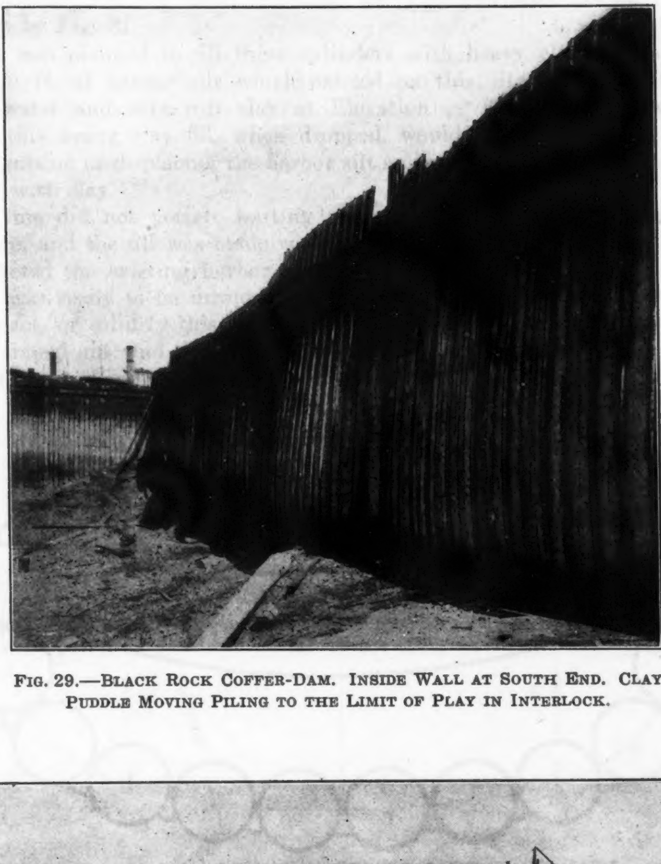


FIG. 29.—BLACK ROCK COFFER-DAM. INSIDE WALL AT SOUTH END. CLAY PUDDLE MOVING PILING TO THE LIMIT OF PLAY IN INTERLOCK.

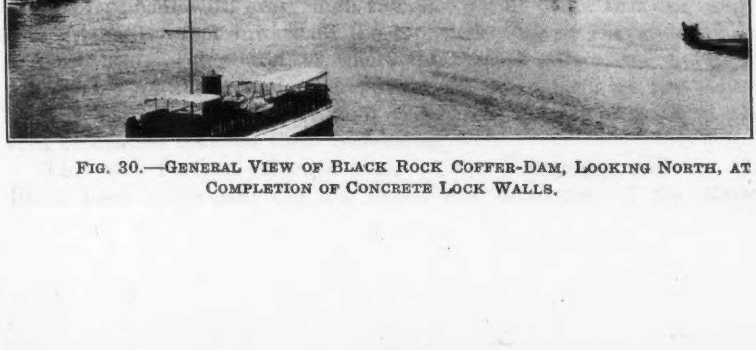
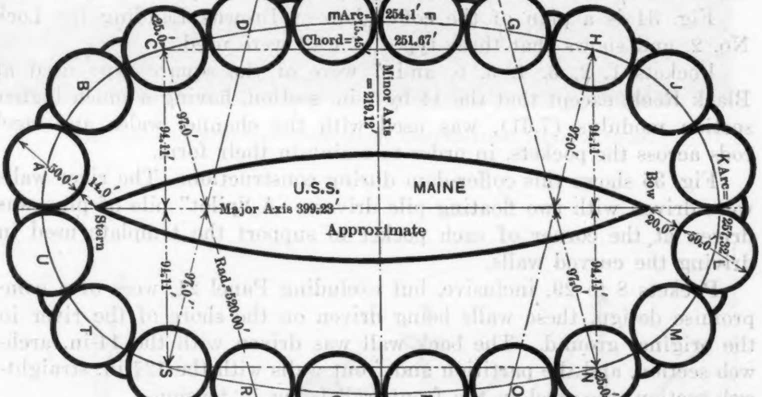


FIG. 30.—GENERAL VIEW OF BLACK ROCK COFFER-DAM, LOOKING NORTH, AT COMPLETION OF CONCRETE LOCK WALLS.

in diameter, on the ends, but this was changed later to the plan shown by Fig. 31. Mr. Boardman.

It was planned to fill these cylinders with heavy clay, displacing the 25 ft. of harbor silt which existed on this site below 35 ft. of salt water and over soft clay at Elevation — 60. It was thought that this heavy clay fill, when dropped, would sink to the clay bed, thus raising or displacing the harbor silt and securing cylinders entirely filled with clay.

Time did not permit waiting to deposit the clay by the dipper-dredge, and the fill was made with a hydraulic dredge. This material blanketed the existing harbor silt in the cylinders. When the coffer-dam was ready to be unwatered, it was found impracticable to settle, compact, or solidify this silt, nor could the water in it be withdrawn or pumped out, and this fill, 25 ft. in height, remained in a semi-fluid condition throughout the work.




COFFER-DAM AROUND WRECK OF BATTLESHIP MAINE

Fig. 31.

For this reason the *Maine* coffer-dam was affected by the water pressures after 15 ft. of water was pumped out, and even showed slight movement with every high tide, the result being that the cylinders gradually crept inward at the top, deflecting at Elevation 40 ft. or more below water level, or where this harbor silt existed as fill in cylinders. As the cylinders crept inward, the length of the center line of each became shorter, and they took an elliptical form, the area of contact between them increasing.

This was further evidence that, as the diaphragm walls in the Black Rock coffer-dam did not move, this movement of the *Maine*

Mr. Boardman.

coffer-dam cylinders tended to create diaphragm walls, and led to the conclusion that, when further coffer-dam construction of this type was required, the design could be much improved by adopting the better parts of each, for the form or shape of the cells or pockets. This is clearly exemplified in the plans for the Forty-sixth Street Pier coffer-dam designed by Mr. Staniford as steel sheet-pile cellular cut-off walls.

The stone embankment of this coffer-dam was placed during unwatering so as to maintain a balance of pressures and to take the place of the weight of the water and mud taken out of the coffer-dam. The base of this stone embankment, being on harbor mud, could not be depended on for a passive pressure in the same manner as the stone embankment for the Forty-sixth Street Pier coffer-dam.

The United States Government engineers recognized and used the best features of the Black Rock and *Maine* coffer-dams when designing those for the lock and dams in the Cape Fear River, North Carolina.

Fig. 34 is a plan of the coffer-dam at Browns Landing for Lock No. 2, and shows that three types of wall were used.

Pockets 1, 2, 3, 4, 5, 6, and 7 were of the square type used at Black Rock, except that the 14 by $\frac{3}{8}$-in. section, having a much higher section modulus (7.61), was used with the channel wales and steel rods across the pockets, in order to maintain their form.

Fig. 35 shows this coffer-dam during construction. The river walls were driven with two floating pile drivers. A "pilot" pile of pine was driven at the corner of each pocket to support the template used in driving the curved walls.

Pockets 8 to 29, inclusive, but excluding Panel 21, were of a compromise design, these walls being driven on the shore of the river in the original ground. The back wall was driven with the 14-in. arch-web section, and the partition and front walls with the $12\frac{3}{4}$-in. straight-web section, the steel in the front wall being in tension.

Panels 21 and 30 each consist of a single wall, constructed after the design of bulkhead or dock walls, of steel sheet-piling with steel channel wales and steel tie-rods to anchors. This work was carried out most successfully, the concrete slab being placed under water through a tremie, when the water was near the top of coffer-dam. The top of the coffer-dam was at Elevation + 32, and the finished lock floor at Elevation + 1.0. The steel sheet-piling was in 49-ft. lengths.

This dam withstood a head of 30 ft. after it was pumped out the first time. Freshets in the river flooded it five times in one season, without any damage whatever. The river was permitted to flow over the top and into the pit without damage. At low stages the leakage was from 200 to 500 gal. per min., thus requiring a small expense for pumping.

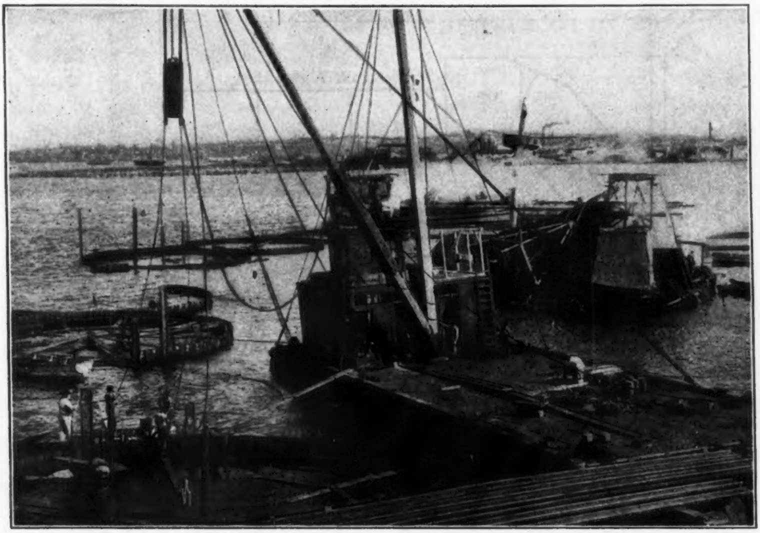
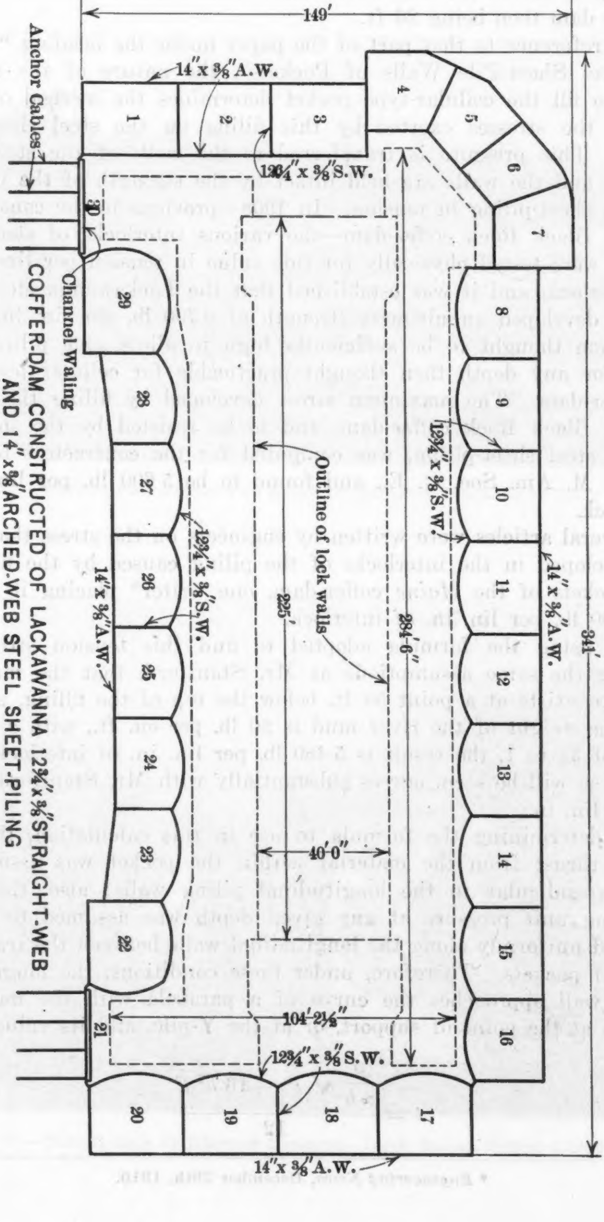


FIG. 32.—CONSTRUCTING THE *Maine* COFFER-DAM, HAVANA, CUBA.

FIG. 33.—THE BOW OF THE *Maine*, WITHIN THE COFFER-DAM.
WATER AT ELEVATION — 10 FEET.

Mr. Boardman.




COFFER-DAM CONSTRUCTED OF LACKAWANNA 12¾"x⅜" STRAIGHT-WEB AND 14"x⅜" ARCHED-WEB STEEL SHEET-PILING

FIG. 34.

Mr. Boardman.

Fig. 36 shows the lock walls under construction, the head of water on the dam then being 26 ft.

In reference to that part of the paper under the heading "Stresses in Steel Sheet-Pile Walls of Pockets", the nature of the material used to fill the cellular-type pocket determines the method of calculating the stresses exerted by this filling on the steel sheet-piling walls. This pressure is transferred to the walls of the steel sheet-piling, and the walls are held intact by the strength of the interlock of this sheet-piling in tension. In 1908—previous to the construction of the Black Rock coffer-dam—the various interlocks of steel sheet-piling were tested physically for this value in tension per linear inch of interlock, and it was established that the Lackawanna steel sheet-piling developed an ultimate strength of 9 700 lb. per lin. in., which was then thought to be sufficiently high to allow such piling to be used for any depth then thought practicable for cellular pockets in a coffer-dam. The maximum stress developed by filling the pockets of the Black Rock coffer-dam, and to be resisted by the interlocks of the steel sheet-piling, was computed for the contractors by J. C. Meem, M. Am. Soc. C. E., and found to be 5 600 lb. per lin. in. of interlock.

Several articles were written by engineers on the stress that would be developed in the interlocks of the piling caused by the filling in the pockets of the *Maine* coffer-dam, one writer* placing it as high as 7 400 lb. per lin. in. of interlock.

By using the formula adopted to find this tension stress, and making the same assumptions as Mr. Staniford, that the maximum pressure exists at a point 58 ft. below the top of the filling, and also that the weight of the river mud is 80 lb. per cu. ft., with a natural slope of 3½ to 1, the result is 5 480 lb. per lin. in. of interlock. This figure, as will be seen, agrees substantially with Mr. Staniford's 5 200 lb. per lin. in.

In determining the formula to use in this calculation, the horizontal thrust from the material within the pocket was assumed to act perpendicular to the longitudinal piling walls; also, the corresponding unit pressure at any given depth was assumed to be distributed uniformly along the longitudinal walls between the transverse walls of pockets. Therefore, under these conditions, the longitudinal piling wall approaches the curve of a parabola with the maximum tension at the point of support, or at the Y-pile, and its value is

$$t = \frac{\frac{p}{8\,h}\sqrt{l^4 - 16\,h^2\,l^2}}{12}$$

* *Engineering News,* December 29th, 1910.

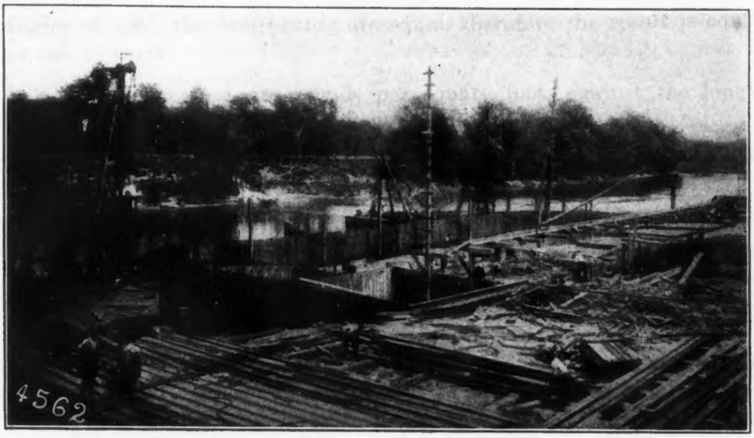



FIG. 35.—COFFER-DAM AT BROWNS LANDING FOR LOCK NO. 2, CAPE FEAR RIVER.

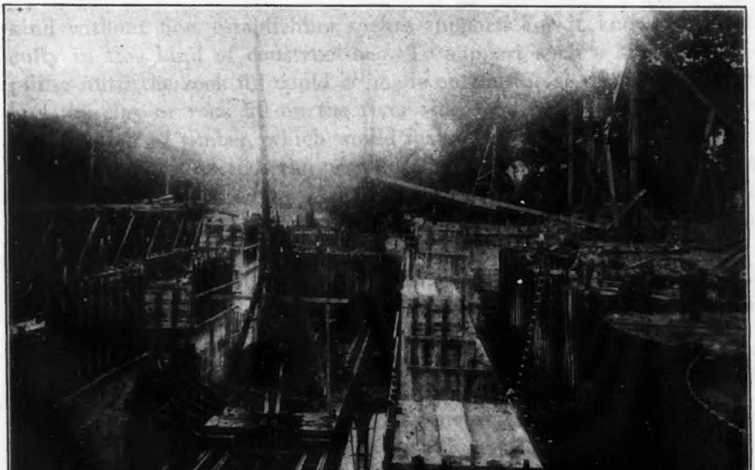



FIG. 36.—COFFER-DAM AT BROWNS LANDING. LOCK WALLS UNDER CONSTRUCTION.

when $t$ = the tension, in pounds per linear inch, in the longitudinal and transverse walls, for the reason that the Y-pile is fabricated with angles of 120°, the components are equal, therefore the result is equal to one of them. Mr. Boardman.

$p$ = the pressure, in pounds per square foot, against the longitudinal walls at the depth considered.
$h$ = the ordinate at the center line of the pocket, in feet.
$l$ = the distance between transverse walls, in feet.

The suggestion has been made that possibly a single wall of steel sheet-piling would have answered for the cut-off wall in a coffer-dam of this type. A thoughtful analysis of this idea will lead to its abandonment almost immediately; first, for the reason that this type of construction, as Mr. Staniford's design shows, requires a rock fill on the inside for stability. Therefore, though it is not impracticable, it is not good engineering to attempt to drive a single wall of steel sheet-piling through this rock fill for such a depth. Again, assuming this cut-off wall to have been constructed before any fill had been placed: this would mean that an assembled single wall, from 60 to 70 ft. in height, would have to stand it vertically, and be secured only by resting on bed-rock and being supported for about one-third of its height in harbor silt. Any one who has assembled a wall of this kind without first establishing secure supports for it knows the difficulty in this kind of construction. To support such a single wall of piling until the rock fill could be made on the inside of the coffer-dam and the clay or rock fill on the river side, would require a temporary construction of timber, which would make the cost of establishing this timber support exceed the cost of building the cellular type of cut-off wall as adopted by Mr. Staniford, which, as has been proved, had sufficient base so that it was stable within itself during the filling of the pockets and during construction. It is readily seen that a cut-off wall of cellular type filled with harbor silt is absolutely water-tight, and must have an advantage in this respect over a single wall of steel sheet-piling.

The conditions at the Forty-sixth Street Pier, when thoroughly analyzed, show that the bed-rock sloped toward the river and had a very uneven surface, with little clay overlying it, and with harbor silt above it; and that this silt was the only available fill for the pockets. These conditions, and also the extreme depth of the coffer-dam, lead to the belief that Mr. Staniford adopted the best plan for this construction. The remarkable rapidity of its construction and the phenomenal success of the entire coffer-dam are proof positive that this is true.

Mr. Black.

WILLIAM M. BLACK,* M. AM. SOC. C. E. (by letter).†—The writer has taken great interest in this coffer-dam, as it was on his recommendation that the rock excavation in the dry was finally resolved upon, and also because there are one or two points made in Mr. Staniford's paper regarding the work of raising the *Maine*, which should be corrected. When the piling of the cylinders of the *Maine* coffer-dam was drawn, it was found, much to the surprise of those in charge, that, as well as could be judged, there had been, in general, no movement of the lower ends of the piles forming the cylinder elements. In general, no movement of the lower end of the piling could be observed at all, but, in one or two of the cylinders, it is believed that some of the piles forming the inside elements of some of the cylinders might have been driven farther down into the clay by the cylinder movements. In general, the movement was a bending of the cylinders without affecting the level of the base.

* Washington, D. C.

† Received by the Secretary, April 9th, 1917.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS

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## DISCUSSION ON PROGRESS REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*



*** Presented at the Annual Meeting, January 17th, 1917.**

## (1) GENERAL PRINCIPLES.

By Messrs. E. A. Stevens, Clifford Richardson, Edward Whitwell, E. W. Stern, W. H. Fulweiler, Walter Buehler, W. M. Kinney, J. A. Johnston, C. D. Pollock, C. E. Mickey, Prévost Hubbard, George E. Hemstreet, Samuel Whinery, T. M. Ripley, James W. Routh, C. R. Allen, Jr., Will P. Blair and Maurice B. Greenough, and William C. Perkins.

Mr. Stevens.

E. A. Stevens,* M. Am. Soc. C. E. (by letter).†—The Committee is to be congratulated on its Progress Report. The field is very large, and presents many difficulties. In many cases it becomes necessary to use the best available material, even though it is not in all ways suitable; in others, none but the best obtainable can be considered. It is naturally impossible to cover such essentially different conditions under concise and definite descriptions.

Without any desire to find fault with the admirable work of the Committee, but to place before it some of the difficulties that arise in applying the policy outlined in the report to one field of work, the following is submitted for consideration.

*Grades.*—On grades as steep as 12% it can hardly be admitted that good practice admits the use of broken stone or gravel. The length of gradient, of course, is a factor. Not even the best of care can prevent the formation of ruts, especially after a snowfall. In such cases, the erosive action of the melted snow will quickly cut ruts in the road metal at such a grade. In the case of long runs, this action becomes destructive. It must be recalled that the greatest crowns recommended—1 in. for gravel and ¾ in. for macadam—correspond to cross-grades of 8 and 6 per cent. At 12% grade, therefore, water will tend to follow the road instead of draining from it, and this tendency is encouraged by the slightest longitudinal depression. The maintenance of earth shoulders also becomes expensive.

It cannot be denied that many gravel and macadam roads are being maintained at excessive grades. In the writer's experience, there are two reasons for this: first, the cost of maintenance for the steep grades, apart from the rest of the road, is not known; second, means are not available for a suitable surfacing, such as hill-side brick or stone block. Even at the cost of a temporary slighting of the rest of the road, it would seem wise to consider paving stretches of any great length over 8% in grade with block from gutter to gutter.

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* Hoboken, N. J.

† Received by the Secretary, January 19th, 1917.

The other grades given are open to some criticism. There seems to be little or no reason for the difference allowed between bituminous surface and bituminous macadam. The actual surfaces of these types are the same, they differ in strength. It would almost seem as if "bituminous surface" and "bituminous concrete" had been transposed in printing. For the latter, which is much more slippery than "bituminous surface", though it affords a better foothold either for horse or motor vehicle tires than sheet-asphalt, 6% is surely a maximum grade. Mr. Stevens.

*Width.*—For the reasons set forth in the report, attention should be drawn to the need of increased width on the inner sides of curves, and, where speeds are high, to the inadvisability of change in cross-section so as to "bank" the curve. These features promote both safety and economy.

*Thickness.*—Too much weight cannot be given to the principle that the bearing power of the sub-base is an important factor in determining thicknesses of road surfaces.

*Crown.*—Good road gravel furnishes about the best surface for high speeds, up to its load-bearing capacity. For high speeds, a crown of 1 in. per ft. becomes absolutely dangerous and highly uneconomical. As pointed out, gravel is often used on roads where speeds are low; even then, if well maintained, ¾ in. seems to be as high a crown as it is wise to use.

CLIFFORD RICHARDSON,* M. AM. SOC. C. E.—This report is a valuable contribution to the literature of the subject, but is open to criticism in some respects. Mr. Richardson.

It is stated that:

"Certain materials, or results of using materials, for highway surfacings will be unsatisfactory outside of certain limits of grades. Conservative practice has fixed the maximum limits for satisfactory results with grades, as follows:

| Kind of Roadway. | Maximum Grade. |
|---|---|
| * * * * * * * | * * * |
| Bituminous surface | 6.0% |
| Bituminous macadam | 8.0% |
| Bituminous concrete | 8.0% |
| Sheet-asphalt | 5.0% |
| * * * * * * * | * * *" |

It would appear that the Committee is composed entirely of eastern men, and that they are not informed in regard to common practice in the West, especially in regard to sheet-asphalt pavements, which has extended over many years. Mr. Tillson, however, a member of the Committee, in his book "Street Pavements and Paving Materials",†

* New York City.

† Second Edition, p. 233.

Mr. Richardson. cites instances where the grade permitted for pavements of this type has been as high as 17%, as shown in the following quotation:

"One of the objections made to asphalt is on account of its slipperiness and the liability of horses falling when they come off from a rough stone surface to the smooth asphalt. There is some reason for this, but as asphalt pavements increase in quantity, horses will become more accustomed to them and learn to adapt themselves to the smooth surface. Asphalt itself, contrary to the general belief, is not slippery. It is smooth, and any soft substance upon a smooth surface makes it slippery.

"Asphalt pavements should be kept clean and then there will be less trouble on account of horses slipping. Asphalt is much less slippery when dry than when slightly damp or moist. It is well known to truckmen that horses travel on a smooth pavement much more easily during a heavy rain than in a drizzle. A certain amount of street detritus must collect on any smooth pavement, and when rain falls in a quantity sufficient to wet it only rather than wash it clean, it must be slippery to a certain extent.

"The question as to what is the steepest grade on which it is safe to lay asphalt has received a great deal of study. When the material was first introduced grades of four per cent. were considered prohibitory, and very little was laid on those exceeding 3 per cent., but practice soon showed that this was too conservative a view, and as a result pavements have been laid successfully and quite frequently on grades as high as 7 and 8 per cent., and in Scranton, Pa., there is a portion of one street that has a grade of 12½ per cent. It was said to have been placed on this particular block for the sake of preventing traffic, but, strange to say, it has not done so, and the City Engineer says that after several years' use no great trouble has been experienced with it.

"Fig. 16 represents a profile of a portion of Bates Street, Pittsburgh, Pa. This shows that the elevation of the grade increased from 188.21 at the property line to 209.63 at a point 200 feet distant, making an average rise of 10.7 per cent. Instead, however, of making a uniform grade, these points were connected by a vertical curve, making in one section a grade of 17.1 per cent., and in the first 80 feet the minimum rate is 12.4 per cent. This street is paved with sheet asphalt, and without doubt has the steepest grade of any street in the world paved with that material."

The speaker, in "The Modern Asphalt Pavement" has called attention to the entirely satisfactory way in which sheet-asphalt surfaces have behaved on grades as high as 10%, as follows:

"Grades of Streets on which Asphalt Pavements may be Constructed.—The general impression has gained ground, very naturally, that asphalt pavements are unsuited to grades of more than 4 to 5 per cent. This is an erroneous conclusion may be seen from the fact that in 1890 an asphalt surface was laid in Washington, D. C., on Thirty-fourth Street, N. W., from M Street to Prospect Street, 275

feet long, the grade of which is 9.74 per cent., and that in Kansas City, Mo., the following streets have been constructed with the grades given. Mr. Richardson.

GRADES IN KANSAS CITY, MO., FOR ASPHALT PAVEMENTS.

| Year Laid. | Street. | Grade. |
|---|---|---|
| 1898 | Jefferson Street, 18 to 20 | 12.5% |
| 1895 | 11th Street, Maine to Wyandotte | 7.5 |
| 1897 | Troost Avenue. 19 to Belt Line | 8.0 |
| 1895 | Central Street, 16 to 17th | 10.0 |
| 1894 | Forest Avenue, Independence to 8 | 8.0 |

"All of these streets are in constant use and are satisfactory except on occasions where a thin coating of moisture has become congealed on the surface. Several of the streets in Kansas City are only paved with asphalt in the center, the sides having a stone or brick surface. Nevertheless, the asphalt surface is universally used in preference to the brick or stone, and appears to be no more slippery even under the most trying conditions. Where a film of ice causes the asphalt to be slippery, traffic is diverted to other streets with lighter grades, rather than to the brick or stone. As a matter of fact the limiting conditions in determining the extent to which the steepness of a grade will prevent the use of an asphalt surface mixture will depend entirely upon the climate and the nature of the traffic which uses the street. Eight per cent. is not an excessive grade under ordinary eastern conditions, while in a climate like Seattle, Wash., a 10 per cent. or 12 per cent. grade is quite possible."

In addition to the foregoing data the speaker would say that in 1906 a sheet-asphalt surface was laid on Miami Street, Wabash, Ind., on a grade of 9½%, which is in constant use, and was reported to him when he visited that locality in 1911, as being entirely satisfactory and not more slippery than other smooth surfaces.

It would seem, therefore, that the Committee has been ultra-conservative in the limits which it prescribes for maximum grades of asphalt surfaces.

The Committee states that:

"The thickness of the pavement or surfacing, of course, will be dependent largely on its type, but it will also be affected by the presence or absence in the construction of an artificial foundation, and, in fact, on the character and ability of the base on which the surfacing is to rest."

Apparently by "ability" is meant "stability".

The terms "artificial foundations" and "artificial surfacing" seem to be very loosely used, or without proper definition. As it appears to the speaker, all surfacing is artificial, and the same is the case with foundations, with the exception of subsoil foundations.

In the construction of concrete foundations it is stated that: "The thickness may be varied sometimes between the center of the roadway and the sides."

It may be assumed, following ordinary practice, that this recommendation would refer to a lesser thickness at the sides than in the

Mr. Richardson.

center. This brings up a point to which attention may well be called. In the speaker's opinion most of the longitudinal cracks in concrete surfaces are due to the fact that there is no proper lateral support for the crown of Portland cement concrete surfaces. They should be built with a greater thickness at the sides than in the center, in order to form an arch which will give the surface greater stability. In city streets, of course, the curb aids in this, but in country roads it is a very great mistake to place concrete without any lateral support. Longitudinal cracks are the natural sequence of omitting this. The Committee apparently realizes this when, in considering the matter of "Shoulders and Gutters" it states:

"The line or strip of contact between a cement-concrete roadway and the flanking material of the shoulders being the zone of weakness under traffic, it is important to accommodate the traffic and to protect the roadway as well as the shoulders from the formation of ruts along this line."

Attention should be also called to the fact that, in the construction of concrete, the character of the sand in use should receive as much care as the cement. The former should be tested, as well as the latter.

The provision that

"Whenever comprehensive specifications are to be prepared, so as to admit a variety of types of bituminous materials, separate specifications, as may be necessary, should be prepared for each type"

is an important one. It has been evident for some time that a blanket specification covering all types of bitumen is entirely unsatisfactory. The Committee is to be congratulated on recognizing this fact and bringing it to the attention of engineers.

In describing the material for broken stone roads it is provided that this "should be clean, rough-surfaced, sharp-angled, etc."

What is meant by "rough-surfaced"? The fragments should certainly not be "sharp-angled", but very obtuse. The use of such a term is unfortunate.

The authors of the report seem to realize that there is no accurate correspondence between the size of the opening in a screen and that of the material which it will pass, and that this size is very largely dependent on the angle at which the screen is placed and on the rapidity with which it revolves. For these reasons, confusion may exist between the commercial designation of the size of the stone and that determined in the laboratory.

The difficulty with reference to specifications mentioned by Mr. Hemstreet can be illustrated by a parallel case. If it is merely specified that an ideal cement be used in concrete, without differentiating between the natural, the slag, and the Portland cements, it would be a

parallel state of affairs. Portland cement is recognized as being in an entirely different class from the others, and, if it is wanted it is specified. Why not have alternate specifications with regard to the asphalts, which also differ in source and perhaps in quality. Mr. Richardson.

The quantity of bitumen used in a broken stone road would depend largely on the character of the stone. For instance, take two sheets of glass, one of polished and the other of ground glass, the latter, on account of its surface, will naturally carry a larger quantity of bitumen than the polished glass; and the same principle applies in the relation of bitumen to broken stone.

On one or two occasions the speaker has called attention to the fact that the English practice in laying blocks of Baltic pine seems to be most desirable. It is the custom to place the blocks on a Portland cement foundation and then fill the lower half of the interstices between the blocks with coal-tar. Water at the bottom of the block causes the damage; it comes from the bottom rather than from the surface. Therefore the bituminous material should be poured in first, in order to water-proof the joints at the bottom; then the Portland cement-grout should be added, and that prevents the bleeding of the coal-tar under the summer sun, and makes an excellent form of construction.

EDWARD WHITWELL,* ESQ. (by letter).†—Referring to Table 1, it is noted that the recommendations fall distinctly short of those called for in European practice, particularly British, and especially with regard to "broken stone" and "macadam." The experience of the English road experts has led to their minimum being quite 30% higher than that of the Committee. As a matter of fact, the Roads Board, which now controls the whole of the main highways in Great Britain, arrived at a table of suitable standards, variable of course to traffic requirements, which fully coincides with this higher figure. Mr. Whitwell.

E. W. STERN,‡ M. AM. SOC. C. E.—In connection with the question of grades, the speaker's observations have led him to believe that one bituminous surface is quite as slippery as another. Even the asphalt block pavement which originally was supposed to be less slippery for horses, has, under automobile traffic, become almost like sheet-asphalt. The oil drippings from the automobiles work into the surface of the pavement and soften it, and the traffic irons it into a homogeneous surface which is quite slippery. It is often difficult to distinguish the joints. The speaker, therefore, believes that all the bituminous surfaces should be classed together as regards their slipperiness. Mr. Stern.

* President, Institution of Municipal Engineers, London, England.

† Received by the Secretary, January 19th, 1917.

‡ New York City.

Mr. Stern.

As regards limiting the gradients, there are many sheet-asphalt streets in New York City on which the grades are greater than 5%, and they are quite satisfactory. One case in particular is quite interesting; namely, that of West End Avenue, north of 96th Street. There is a grade there of 6 per cent. The pavement is sheet-asphalt. Near-by is 96th Street having a granite block pavement with a grade of 7 per cent. Heavy horse-drawn trucks climb the sheet-asphalt street in preference to the granite, in order to save the difference in grade of 1 per cent. The truck drivers do not seem to mind the 6% grade on the sheet-asphalt, as the horses are shod for it. This street is used all winter by heavily laden horse-drawn trucks.

New York City has many streets on which the grade is as high as 8 or 9%, and the speaker has frequently asked the Board of Estimate for permission to lay sheet-asphalt on these grades, the Board having limited the grade for sheet-asphalt to 5 per cent. If the traffic were altogether horse-drawn, perhaps there might be some excuse for going back to the practice of the old days, but traffic is changing so rapidly to motor-drawn vehicles that it is thought that grades on sheet-asphalt streets could properly be increased.

There are many examples in New York City that confirm Mr. Richardson's statement as regards the preference of drivers for sheet-asphalt. On 110th Street, between Amsterdam and Columbus Avenues, the grade is 5%, there is a granite pavement in the middle of the roadway, and on each side of it there is a strip of asphalt about 8 ft. wide. The speaker has noted that the truckmen in going up that grade invariably take the sheet-asphalt in preference to the granite. On Park Avenue the same thing has been noticed. There are some blocks on this avenue where there used to be strips of granite along the sides with sheet-asphalt in the middle. Truckmen did not take to the granite strips, but preferred the sheet-asphalt; hence, these strips of granite have been replaced with sheet-asphalt in order to reduce the noise, as many complaints had been received regarding this.

With reference to broken stone roadways, the speaker's observations have led him to believe that if such surfaces are used for 12% grades, at the end of every heavy rainstorm a great deal of the surface material will be down at the bottom of the grade. The speaker believes that such grades are altogether too steep for this material for a permanent roadway.

With cement concrete the grades may be much higher. The speaker is informed that in Sioux City, Iowa, there are grades as high as 18%, and in Portland, Ore., as high as 20 per cent. The surface is not nearly as slippery as sheet-asphalt, not being slick, but more like sandstone. The speaker believes that the limit of 8% for this type, suggested by the Committee, can be raised considerably; also, that

for brick with cement-grout filler the limit can be raised, for the same reasons as given above. Mr. Stern.

It is really unfortunate that wood block is the most slippery pavement of all because it has many valuable qualities. It should not be used on grades as high as 4 per cent. Though there are some strips with such grades in New York City, continual complaints from the police and others are received, and very often there are requests for the removal of the wood block pavements and the substitution of granite. The speaker believes that 2% should be the maximum grade for wood block.

Asphalt has the advantage over wood in not being slippery in really wet weather. Wood block is always slippery in wet weather; whether there is a drizzle or a heavy rain.

The speaker takes issue with the Committee's recommendation that separate specifications should be prepared for each type of bituminous material. It is wrong in principle. The tendency in all engineering specifications for materials to-day is to reduce them to some common denominator, if possible. In purchasing structural steel, engineers want certain results and ask for them, and it is the same as regards Portland cement, etc. In bituminous materials, engineers are interested largely in the cementitious value, adhesiveness, and susceptibility to temperature changes; and a single specification covering those properties should be drawn.

As regards the use of joint fillers in stone block pavements the Committee recommends that sand should never be used alone as a joint filler. The speaker is fully in accord with this.

The recommendation regarding bituminous mastic for joint filling, as being an improvement over the customary practice of using bituminous material alone for this purpose, is to be highly commended. In the Borough of Manhattan, City of New York, some experiments are now being made with both the tar and the asphalt joint filler by taking a certain street and filling the joints with tar for half a block, and with asphalt for the other half. An attempt is also being made to obtain the effect of a mastic joint by putting in as much fine sand as possible. The speaker believes that the more sand one can put into the bituminous material, provided it is thoroughly mixed, the better the joint will be.

One matter which should be brought to the attention of the Committee is a desirable joint for block pavements on steep grades. It is a puzzling question. If laid with open joints and with a bituminous filler, the blocks round off in time under heavy traffic, and get noisy. In time the filler works out and leaves the joints open, and then, of course, there is a better foothold for horses. A joint made with Portland cement grout is too slippery, though it has been

Mr. Stern. reported that some stretches have been made with a ¾-in. raked joint of that material. The speaker has noticed, in some of the technical papers, a description of another type, used on steep grades, namely, a block pavement laid toothed or saw-shaped. It is suggested that the Committee investigate the matter a little further, as it is extremely important.

Mr. Fulweiler. W. H. FULWEILER,* ASSOC. M. AM. SOC. C. E.—Attention is called to what appears to be an error in maximum grade, particularly as the maximum for cement-concrete is given at 8%; bituminous surfaces 6%; and brick with cement-grout filler 6 per cent.

In West Broadway, Baltimore, Md., there was a grade that approximated 8% on a cement-grout brick pavement. That surface was usually satisfactory, but at certain times heavily loaded teams could not get up the grade comfortably, and were compelled to make a detour of several blocks. An experimental bituminous surface was laid on that street, and the teamsters no longer had to make a detour. They could go straight up. The speaker is of the opinion that the bituminous surface, when applied properly, should be given at least the maximum grade of the brick.

A certain cement-concrete road in New York State had some fairly stiff grades. During the first winter after it was finished, teamsters had great difficulty in climbing some of the grades, and were compelled to drive on the shoulders, to their great detriment. However, after a bituminous surface had been applied on this concrete, the grades were taken with ease.

The last paragraph under "Materials" opens up an exceedingly important phase of the whole discussion of specifications. It is the speaker's opinion that, due to ignorance of the fundamental requirements of the materials used, many specifications are not specifications at all, but are descriptions. They are descriptions, in more or less technical language, of certain commercially available products. They bear no resemblance whatsoever to specifications for iron or steel, or for cement, because in such cases engineers have agreed that they require for the work to be done certain physical effects; and they proceed, by means of tests, to secure in the material those particular physical results.

Now, in bituminous materials, unfortunately, knowledge of the requirements and of the physical phenomena underlying those requirements is so meager at present that engineers are not able, either to develop tests, or to come to an agreement as to just what qualities are required, or what results those qualities must present. Thus they must use descriptions of certain available grades of material that

* Wallingford, Pa.

experience or inference—frequently inference—would indicate would probably be satisfactory for the purpose. Mr. Fulweiler.

One of the greatest works that this Committee could do would be to endeavor to collect certain data, such as would enable it to present to the Engineering Profession at large a clearer idea of the qualities that those materials should possess, and an idea of the results required for specific purposes. It is a confession of weakness to recommend such specifications, and it is an unfortunate state of affairs. However, the speaker believes that the research work now being done in many parts of the country will eventually result in a general agreement as to the qualities necessary and the tests whereby those qualities may be expressed in actual figures.

WALTER BUEHLER,* M. AM. SOC. C. E.—The speaker believes that the slipperiness of wood block, mentioned by Mr. Stern, is a matter which can be taken care of very easily by proper construction. If a bituminous filler is used and is applied with squeegees, and this is followed by an application of sharp sand, which is allowed to remain under traffic for 2 or 3 months, and sprinkled so that it will not be blown about and become a nuisance, the rough surface thus obtained is far from slippery. At the present time, the City of Minneapolis has about 70 miles of creosoted block pavements, nearly all of which are of Southern pine, laid in this way, and to the speaker's knowledge, there has never been a complaint of slipperiness. Mr. Buehler.

In Table 1 of the report, under "Thickness of Wearing Course," the minimum for wood block pavement is given at 3½ in. On the business streets of a great many small cities in the West, and on the residence streets of the larger cities, 3-in. blocks have been and are used quite successfully.

There is a new form of construction in which the sand cushion is eliminated entirely and the blocks are placed directly on the concrete which has been prepared with a smooth surface and painted with pitch, with blocks as shallow as 2½ in. This construction is now being advocated by some of the wood block manufacturers, who base their recommendations on experience with the shallower blocks. The speaker would not recommend, at this time, a minimum thickness of 2½ in., but would suggest that the report be amended to read 3 in., instead of 3½ in.

It is the speaker's opinion that sand should never be used for a filler in wood block pavements. It is true that there are some conditions under which its use may result successfully, for example, in narrow streets having very heavy traffic; but, under ordinary conditions, a bituminous filler should be used. It is a mistake to ram the blocks together so closely that it is impossible to put in a filler.

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* Chicago, Ill.

Mr. Kinney. W. M. KINNEY,* ASSOC. M. AM. SOC. C. E.—In recommending for cement concrete surfaces a maximum grade of 8%, the Committee has apparently failed to take cognizance of the successful use of pavements of this type on grades as steep as 16 per cent. In Duluth, Milwaukee, Seattle, Chehalis, Washington, and Sioux City, grades of more than 8% are in constant use, the following extract from a letter from T. H. Johnson, City Engineer, of Sioux City, being typical of the successful use of concrete on such grades:

"The idea that concrete is not adapted to grades above 6% is wrong. We have in use 5½ miles ranging from 6 to 9%, 2 miles from 9 to 12%, 0.2 mile from 12 to 15%, and 0.2 mile of 16 per cent. All of this is in constant use, and there is every indication that it has preference over other kinds on similar grades."

It is difficult to recognize the fine distinction drawn between various types of pavement under the heading "Crown Recommended" in Table 2. It would seem that the minimum crown recommended is a function, not of the material of which the pavement is constructed, except in the case of gravel and macadam, but rather of the possibilities as to workmanship in finishing the surface to the desired contour. It is difficult to understand why it should be possible to construct a brick or asphalt pavement to a more accurate contour than a cement concrete pavement; in fact, the contrary is rather the case, and the actual measurement of a number of jobs indicates that concrete is usually constructed to a more perfect contour than either of the other types mentioned.

Mr. Johnston. J. A. JOHNSTON,† M. AM. SOC. C. E.—The thickness of the "artificial foundation", mentioned in the report, is somewhat misleading, though the foot-note states: "Not including extraordinary provisions such as V-drains or sub-base courses." It is assumed, therefore, that this thickness of "artificial foundation" is merely the bottom course of such surfacing material as may be used. The speaker thinks the report should emphasize the absolute necessity of a sufficient foundation to support such surface as may be built. Given a proper foundation, less surfacing material may be used than has been recommended as the minimum.

Many roads are being constructed in Massachusetts with as little as 2 in. of crushed stone and bitumen on top of a base of stone fragments from ledges and walls. This base may vary from an average thickness of 6 to 8 or 10 in., with the voids filled with sand. Such roads have been in place for 5 years, have caused very little trouble, and carry a traffic of about 500 vehicles a day, with a very

* Chicago, Ill.

† Springfield, Mass.

small maintenance cost. Therefore the minimum thickness might be even less than that recommended in the report. Mr. Johnston.

It is noted that the report gives for bituminous macadam a minimum thickness of 3 in. The speaker, as he has previously stated, is using as little as 2 in., and is getting good results.

C. D. POLLOCK,* M. AM. SOC. C. E.—Attention is called to Table 1 in which the thickness of the artificial foundation for stone block is given as from 5 to 12 in., whereas, for the remaining materials, it varies from about 3 to 8 in., the latter being the maximum. The speaker questions the propriety of using a greater thickness than 8 in. for the foundation, as he has examined foundations over which very heavy loads have passed, in fact, where the pavement itself was badly damaged, but the foundation was of good quality and 6 in. thick, and there was no evidence of its destruction or damage. It would seem that a depth of from 5 to 8 in. would be ample; more than that would be a waste of money. Mr. Pollock.

In the second paragraph from the bottom of page 1617, it seems to the speaker that there is an error in regard to the statement as to the mixture of sand and cement for grout filler: "Where the blocks are of less resistant material and conditions demand economy, a 1:1 mix of cement and sand may be found satisfactory." This statement appears to duplicate the other. Possibly a 1:2 mix was intended.

C. E. MICKEY,† ASSOC. AM. SOC. C. E.—The speaker takes issue with the last paragraph under the heading "Materials:" Mr. Mickey.

"Whenever comprehensive specifications are to be prepared, so as to admit of a variety of types of bituminous materials, separate specifications, as may be necessary, should be prepared for each type."

It is not necessary to specify for the products of different brick plants, cement plants, or any plant furnishing any particular material used in the construction of pavements. If, as Mr. Stern has said, a specification is adopted, setting forth the requirements and suggesting the conditions which may exist for a pavement that is being considered, that would be sufficient. There are a number of material men who are trying to dominate city boards, by means of classifying specifications, particularly on asphalt. They do this by submitting specifications to the village or city boards on Class *A* asphalt and Class *B* asphalt. One of the large asphalt companies, now doing business in America, usually sees that it is to its advantage to have Class *A* adopted, which sets forth its brand. In Nebraska, Iowa, Kansas, and Missouri, engineers are getting away from this classified specification, and are using a single specification for asphalt and asphaltic cement.

* New York City.

† Lincoln, Nebr.

Mr. Mickey. The speaker cannot see that it is any more advantageous to adopt Class *A* for one asphalt than it would be to adopt Class *A* for one particular brick, providing that the specifications set forth the quality and kind of materials or pavement wanted.

In answer to Mr. Richardson on the cement proposition, the American Society for Testing Materials has set forth the requirements for Portland cement, and these are being used all over the country in specifications for that material. The physical properties are specified, and engineers are pleading ignorance when they specify in any other way for any other material.

Mr. Hubbard. PRÉVOST HUBBARD,* ASSOC. AM. SOC. C. E.—Mr. Mickey has brought up a very important point for consideration, and one on which the Committee might well express an opinion, either in an appendix to this report, or in a future report. The matter of alternate type specifications and blanket specifications has been greatly misunderstood and abused, so far as their application is concerned. The alternate type specification is prepared, primarily, to secure satisfactory materials of different types. If they are included in the same specification, these types should presumably be equal in value, so far as ultimate results are concerned. The selection of the type, then, if two or more types are included in the specifications, should not be at the option of the engineer, but at that of the contractor. In other words, the type which could be procured for the lowest price should be accepted. Now, as a matter of fact, that practice is seldom, if ever, followed, and the alternate type specification is made the basis for forcing into use certain trade products, which may, perhaps, and usually do, cost more than the cheapest type of equivalent value.

Nothing has been said about the construction of bituminous roads from a very soft coraline rock. In Florida this rock has been used successfully in roads of bituminous macadam types, whereas, in New York State, it would be suitable only for hot surface treatment.

Mr. Hemstreet. GEORGE B. HEMSTREET,† ESQ.—The speaker does not agree with the first speaker, who took issue with the Committee. His proposition is understood to be that he would dump all kinds of bitumens into one pot and have one specification to cover it all. In the United States the term "bitumen" includes coal-tars, water-gas tars, and all the varieties of asphalt. By making one specification to cover all classes of bitumen, the specification would be so wide that practically all those substances could be used under it. Now—looking at the matter from the buyers' standpoint—when the speaker buys he tries to draw his specification just as closely as he can, and he certainly has to draw alternate specifications. If he wants a bitumen that comes from what is

* Washington, D. C.

† Supt., Hastings Pavement Co., Hastings-on-Hudson, N. Y.

known as a natural source, that is, a natural asphalt, he cannot draw a specification which will cover that and also cover an asphalt made from a semi-asphaltic or perhaps a mid-continental oil; and if he wants a paraffin base residuum, he cannot draw a specification that will cover that and at the same time cover asphaltic residuum. The speaker does not see how engineers can get away from having the bitumens arranged in different classes, if their specifications are to be so close that they will really mean anything. Mr. Hemstreet.

SAMUEL WHINERY,* M. AM. SOC. C. E.—The time should and probably will come, in all branches of engineering work, when engineers will be able to specify ultimate qualities or results wanted, rather than the qualities in detail of the materials to be used. If they knew and could specify adequately the qualities requisite in bituminous paving cement for the various types of bituminous pavement and road surface, the kind or quality of bitumen entering into its composition would not be important. Mr. Whinery.

As long as they think it necessary to specify the qualities of the bitumens that may be used in making up the bituminous cement, however, it seems necessary to provide separate or alternate specifications for each. The different bitumens on the market vary so widely in their individual, physical, and chemical characteristics that any attempt to frame a general specification sufficiently broad to admit all or several of them is likely to be so indefinite as to admit inferior materials or to produce a final product of inferior quality. It would be neither logical nor fair to a contractor to permit him to use cornmeal and require him to produce from it a high quality of wheat bread.

In the present status of knowledge and experience, it seems to be advisable—if not absolutely necessary—if it is desired in the interests of competition, to permit the use of different bitumens for pavement and road work and to provide separate or alternate clauses of the specifications appropriate or proper for each.

T. M. RIPLEY,† M. AM. SOC. C. E.—In New York State the range of geologic conditions is probably as wide as, or wider than, in any other part of the country. Under the same specification, a stone delivered from one quarry will be entirely different in shape and general character from the material from another quarry; and 2 or 2¼ gal. per sq. yd. for a stone of one kind will give as good results as 2½ or almost 3 gal. per sq. yd. with other kinds of stone, and the first stone would not have absorbed 2¾ or 3 gal. per sq. yd. The speaker cannot find definite recommendations as to how much to use, and that has been practically admitted by the Highway Depart- Mr. Ripley.

* New York City.

† Watertown, N. Y.

Mr. Ripley. ment of New York State, for, during the past 4 or 5 years, it has specified from 1.65 to 3 gal. per sq. yd. on a penetration bituminous road. There are roads in good condition to-day that were built with 1.65 gal., and there were other roads built with 2½ gal. which went to pieces in 2 or 3 years. The maximum limit was raised to 2¾ gal. about 2 years ago, and during the past year the speaker has tried to force some as high as 3 gal. per sq. yd. That, as far as the speaker knows, was not due to the character or quality of the bituminous material, but almost, if not exclusively, to the character of the stone.

Mr. Routh. JAMES W. ROUTH,* ASSOC. M. AM. SOC. C. E. (by letter).†—By incorporating in its report the fundamental principles on which specifications governing the construction of each of the several types of roads and pavements should be based, the Committee has added materially to the value of its work. Though it is true that most of these principles may be of common knowledge to the majority of the engineers engaged in road or pavement construction, nevertheless, many things of common knowledge are of so common knowledge as to be neglected unless sharply brought to mind from time to time. The statements of the Committee should serve an excellent purpose.

*Sub-Grade.*—Too much emphasis cannot be laid on the proper preparation of the sub-grade. Concrete foundations are a great aid in securing good pavements, but, after all, the function of the concrete is to distribute the load over the sub-grade. If the load must be borne by it, the usual depth of inferior concrete is inadequate for the purpose. Greater thickness and reinforcement are necessary. This additional expense, however, may be avoided by constructing and protecting the sub-grade properly.

The proper construction of the sub-grade includes, not only the careful selection of materials and the securing of uniform compactness, but provision for maintaining the sub-grade in proper condition after the construction of the pavement. One of the essentials in this respect is drainage. It may be thought that the necessity for proper drainage is well recognized. Doubtless this is so, yet consistent violations of good practice in this respect are to be observed in many localities.

With special reference to city streets, it is believed that sufficient attention is not paid to adequate under-drainage. It is believed that many failures of city pavements are due solely to this. The necessity for under-drainage, of course, generally arises only on residence streets, where there are parkways and lawns from which rainwater may gain access to the sub-grade, and it is always possible that the ground-water level may be high. The latter point is frequently overlooked in the construction of city pavements. The writer has in mind

* Rochester, N. Y.

† Received by the Secretary, January 19th, 1917.

one particular instance in which tile under-drains were omitted in the construction of a pavement, because the inspector considered that the chances of water "seeping in behind the curb" were small. A casual inspection, however, disclosed the fact that the ground-water level was high, and the soil was such as to make an unstable sub-grade unless proper drainage was provided. The writer believes that better and wider recognition of this feature of pavement construction is needed, and that at least a plain statement of the case should be included in the final report of the Committee. Mr. Routh.

*Expansion-Contraction Joints.*—Under the heading "Expansion-Contraction Joints", the report states that such joints

"At intervals across certain types of pavements, such as brick and cement-concrete, as well as along the curbs, have been used to compensate for more or less unavoidable movement of the pavement slab."

With special reference to brick pavements, there is a marked tendency on the part of some engineers no longer to provide transverse joints; and some very excellent pavements are to be seen which have no such joints. It may also be said that there are pavements to be seen which seem to indicate the need of them. Would it not be possible for the Committee to express an opinion as to the necessity for transverse joints?

C. R. ALLEN, JR.,* ASSOC. M. AM. SOC. C. E. (by letter).†—*Lines and Grades.*—The writer thinks the Committee has placed the maximum grade for broken stone roads too high. Either a gravel or broken stone roadway on a 12% grade is almost sure to be somewhat displaced or damaged by heavy showers, but as, in general, gravel roadways can be more easily repaired, it is thought that they should be left at 12% and broken stone roadways reduced to 10 per cent. Mr. Allen.

The Committee gives two maximum grades for brick roadways, the difference being in the filler used. It seems to the writer that a better line on which to divide the two classes would be as to whether joints are or are not filled flush with the surface. Satisfactory brick pavements have been laid with hillside brick and cement grout up to 12% grades. Also, it is possible to get good results with ordinary brick with grout raked out at the top.

*Width.*—Some consideration should be given to designing narrow roads in such a way that the width may be increased as traffic demands it and funds may be available.

*Drainage.*—There is no reason for making the minimum crown on cement-concrete roadways more than the minimum crown for brick and wood block roadways.

* Albany, N. Y.

† Received by the Secretary, January 19th, 1917.

Mr. Allen.

*Joints.*—The Committee well says, in regard to cement grout, that ample time must be allowed for grout to set before traffic is allowed on the pavement. The writer is thoroughly convinced that more failures are due to the neglect of this requirement than from any other cause. The practicability of this requirement should in many cases decide what filler is to be used. In most cases where cement grout is used for filler, the roadway should be closed to traffic for 3 weeks. This may be somewhat radical, but it would be of value if the Committee would give minimum and maximum limits in place of the indefinite word "ample."

Messrs. Blair and Greenough.

WILL P. BLAIR, ESQ.,* and MAURICE B. GREENOUGH, ESQ.† (by letter).‡—A careful examination of this report leads the writers to believe that, if certain changes were made in its language, the situation would be met more adequately and correctly. Therefore, the more important points, on which that belief rests, are stated briefly in the following for which most serious consideration is asked.

*Grades.*—On page 1612 the report reads as follows:

| | |
|---|---|
| "Brick (cement grout filler)............... | 6.0% |
| "Brick (bituminous filler).................. | 12.0%" |

It is suggested that this portion of the table be changed and amended to read:

| | |
|---|---|
| "Brick (cement grout filler)...............up to | 6% |
| "Brick ('hillside' with cement grout filler).. | 6% and upward |
| "Brick (bituminous filler).................. | 12.0%" |

*Width.*—On page 1613, in the second and third paragraphs, it is recommended:

(*a*) That brick roads should be as wide as, or wider than, the so-called flexible pavements under the same local conditions.

(*b*) It is implied that the rigid edge of a brick pavement is less capable of maintaining its integrity under traffic than a mechanically or otherwise bonded pavement of broken stone.

The writers believe that experience does not support these theories, and that, in justice to the public, they should be altered. Many localities are served economically by brick roads 9 ft. wide, and, if this recommendation is to stand, the result may be to deny to similar localities brick roads which would serve all their functions economically and well.

*Joints.*—On page 1617, the discussion on joint fillers does not meet the situation as regards brick pavements, notwithstanding the very complete exposition of the fundamental requirement of fillers.

* Secretary National Paving Brick Mfrs. Assoc., Cleveland, Ohio.

† Cons. Engr., National Paving Brick Mfrs. Assoc., Cleveland, Ohio.

‡ Received by the Secretary, January 19th, 1917.

Messrs. Blair and Greenough.

The statements are made:

(*a*) That sand should never be used as a joint filler; and subsequently, on page 1633, that "brick laid with sand joints and without an artificial foundation may be used as the first step in road improvement."

The use of sand as a filler is warranted in cases where brick pavements are desired, but where funds are not available for a more costly type. The fact that many thousand square yards of sand-filled brick pavements have rendered satisfactory and economical service for many years precludes light weight being assigned to this filler.

(*b*) The report reads: "As regards the cement mortar joints, the proportions of cement, sand, and water will be affected by local conditions."

A country-wide examination of specifications and practice in joint filling will show that, in the proportions of cement and sand in joint fillers, there is practically no variation one from the other, and that this element of brick pavement construction is the least variable of any.

(*c*) Though several precautionary measures are given for mixing and applying cement grout filler, the writers believe that, unless all the important points are mentioned, the opinion may be created that only those mentioned are the important ones. There are at least eight others, and it is recommended that they be included in order to maintain a proper balance and state the case comprehensively.

(*d*) Continuing, on page 1617, the report states:

"A bituminous filler may be preferred to a cement-grout filler, on account of the lower cost of street-opening repairs, the better foothold provided for horses, and the securing of a more resilient and hence less noisy pavement."

The writers believe:

1.—That a conciliatory attitude toward "cuts and openings" in pavements will tend to keep these unnecessary evils with us longer than an attitude which strongly discourages them.

2.—That brick pavements, cement-grout filled, are the least slippery; that, except on grades greater than 6%, no open joints are required; and that, when required, such joints are best secured by the use of special "hillside" block with cement-grout filler.

3.—More noise is produced by an iron-tired vehicle rolling over open joints and cobbled brick than over a smooth-surfaced, cement-grout filled, brick pavement.

(*e*) On pages 1617-18, the report reads:

"With bituminous joint fillers, care must be taken to select materials which will not be too brittle in cold weather and so chip out from the joints under traffic, and which will not be so soft in hot weather as to flow out of the joints between the blocks."

Messrs. Blair and Greenough.

1.—It is submitted that no bituminous filler has yet been devised which can meet this requirement; and that experience clearly shows this fact.

2.—The unsanitary features of open joints as collectors of dirt and refuse should be brought out, as well as the fact that open-jointed pavements are cleaned with difficulty, if at all, and at all events at much greater expense.

*Expansion-Contraction Joints.*—The report encourages the use of transverse expansion joints in brick pavements. Such joints are now wholly abandoned in places having the most extensive experience in the use of brick pavements, and this relates to the past 10 or 15 years. Brick pavements are capable of withstanding stresses produced thermally with a large factor of safety.

Mr. Perkins.

WILLIAM W. C. PERKINS,* M. AM. SOC. C. E. (by letter).†—At the meeting in 1916 the writer stated that he believed the report of the Committee should be broad and yet describe in detail the various types of materials for road construction, the proper selection and testing of such materials, and the methods of obtaining higher types of construction. In other words, he believes that, owing to the importance of road construction, it is time for this Society to prepare a report so comprehensive that engineers could use it as a framework on which to write their specifications.

The present report is more complete than that of 1916, yet it does not go far enough in describing the materials and their proper construction.

The writer feels sure that the Committee has spent considerable time in preparing the report, and it must be remembered that the members of the Committee are busy men and give their time voluntarily to the Society; but he is certain that, if the Committee, during the coming year, will hold open sessions and call on engineers who have made special studies and investigations of the various materials and methods of construction, these engineers will gladly give the results of their studies and investigations.

The writer believes the Committee should also investigate, by direct visit to the work, the new methods of road construction, thereby obtaining first-hand information as to the many improvements being made in all types of construction. If the powers granted to the Committee do not allow this, the writer trusts that the Board of Direction may see the importance of the work, and widen the powers of the Committee, so that its final report will be considered by engineers as the best practice in preparing specifications and in the construction of roads and highways.

* Conneaut, Ohio.

† Received by the Secretary, January 19th, 1917.

## (2) GRAVEL ROADS.

BY MESSRS. WILLIAM GOLDSMITH, E. A. STEVENS, AND C. E. MICKEY.

WILLIAM GOLDSMITH,* ASSOC. M. AM. SOC. C. E. (by letter).†—On page 1621, with relation to gravel roads, the following statement is made: Mr. Goldsmith.

"With gravel such as quartz, the cementation of which is low, a highly cementitious void filler is desirable, and a moderate quantity of clay or loam may be permissible."

Relative to this subject, the writer had some experience with gravel road construction in the Philippine Islands, where, in Pangasinan Province alone, more than 100 miles of gravel roadway were constructed.

These roadways look like macadam, and, with proper maintenance, hold up as well. The gravel was obtained by screening material from the stream bed, all passing a 1-in. screen being used. It was mostly quartz, and had very low cementitious qualities. Many methods were tried in order to obtain the best possible roadway. This resulted by adopting a crown of 1 in. to the foot, and using a heavy clay filler to fill the voids and hold the gravel together. Instead of a "moderate quantity" being used, as recommended in the report, the filler contained about 20% of clay. It was found that a less quantity would not hold the gravel together. The high crown caused the water to run off before it could soak into the roadway, and disintegrate it.

It is suggested, therefore, that further experiments be made along these lines, and that the words "moderate quantity" be stricken out, and the words "variable quantity, depending on conditions", be inserted.

E. A. STEVENS,‡ M. AM. SOC. C. E. (by letter).†—The term "gravel roads" covers a very wide range of materials. Specifications based on physical analyses are not satisfactory. For instance, there are found in the New Jersey coastal plain certain deposits, locally known as "buckshot gravel", little, probably not more than 20%, of which would be retained on a ¼-in. screen, yet the material in many of these deposits is excellent. Mr. Stevens.

As to gravels, they differ so widely, both in their constituents and grading, that, in their selection, it seems wise to rely on the old principle that "the proof of the pudding is the eating". Above all things,

* New Hampton, N. Y.

† Received by the Secretary, January 19th, 1917.

‡ Hoboken, N. J.

Mr. Stevens. avoid specifying a gravel from a given deposit. Gravel from the same pit will often show amazing variations.

In the case of these roads, the experience everywhere in New Jersey is that a sandy sub-base requires a gravel rich in clay, and a base carrying clay requires gravel with a minimum of clay and rich in sand. Whatever may be true of other gravels, those of the New Jersey coastal plain are not suited for bituminous dressings. Oils, whether asphaltic or non-asphaltic, are actually destructive. The results with cut-back asphalts and tars have not been found durable enough to warrant the expense involved.

Mr. Mickey. C. E. MICKEY,* ASSOC. AM. SOC. C. E.—On page 1621 of the report, there is the statement:

"With gravel such as quartz, the cementation of which is low, a highly cementitious void filler is desirable, and a moderate quantity of clay or loam may be permissible."

The speaker would like to reinforce Mr. Goldsmith's statement, and to add that a cementitious clay, or a loam of high cementitious or cementing value, be used. Some clays, as well as some loams, have practically no cementing value. It is suggested that the Committee insert the words, "of clay or loam of highly cementitious value."

* Lincoln, Nebr.

## (3) BROKEN STONE ROADS.

By Messrs. H. S. Mattimore, Clifford Richardson, W. H. Connell, and Sanford E. Thompson.

H. S. Mattimore,* Assoc. M. Am. Soc. C. E.—The second paragraph of the report on "broken stone roads" calls for an artificial foundation of clean coarse sand or cinders. The speaker has never had any experience in the use of cinders, but would not recommend the use of clean coarse sand, because he does not believe it suitable. The main object in laying the foundation is to get a stable material. This might be coarse gravel, containing fine cementitious gravel, or mixed sand and clay. Mr. Mattimore

In regard to materials, the report might be qualified further and discrimination be made where a two-course broken stone road is laid.

It is presumed that the test refers to the top course. Usually, in the bottom course, any available material that will stand the roller is used. In regard to the size of the stone on a two-course road, a larger stone has been used with success in New York State, say, one that passes through a 2¾-in. ring, or approximately between 2¼ and 2½ in. The size should vary somewhat with the quality of the stone. Stone which is apt to break under the roller, necessarily would have to be larger in the top course than that of a harder quality. This is especially true of penetration roads.

Clifford Richardson,† M. Am. Soc. C. E.—The speaker suggests that the term "sharp-angled" be not used. The stone should not have sharp, but obtuse, angles. A sharp-angled stone would be regarded as undesirable. Mr. Richardson.

W. H. Connell,‡ Assoc. M. Am. Soc. C. E.—The report states that on an 8-in. road the sizes properly used are as follows: "Bottom or first course, 2¼ to 3½ in.; 3½ in. thick. Second course, 1¼ to 2¼ in.; 2½ in. thick. Third course, ⅝ to 1¼ in.; 2 in. thick." Mr. Connell.

In a great many States, and in localities with which the speaker is familiar, 2½-in. stone is now being used in the top course. It gives better results and lasts longer. It is more difficult to bind the road, but it lasts longer and takes the bituminous surface much better than the 1¼-in. top course.

Sanford E. Thompson,§ M. Am. Soc. C. E. (by letter).‖—Attention is called to the fact that the method of screening proposed for aggre- Mr. Thompson.

* Albany, N. Y.

† New York City.

‡ Philadelphia, Pa.

§ Boston, Mass.

‖ Received by the Secretary, January 19th, 1917.

Mr. Thompson.

gates for broken stone roads is directly contrary to practically universal practice for aggregates for concrete work. The report gives the percentages retained between two sieves, instead of the total percentage passing. If there is any variation in the sizes of the sieves selected, the method given in the report cannot produce comparative results, whereas the total percentage-passing method can be used with any sieves, is suitable for use in drawing curves, and for making mechanical analysis combinations. In view of these facts, the writer proposes, as an amendment to these specifications, that the reference on page 1623 to the Report of Committee D-4 of the American Society for Testing Materials be omitted.

## (4) BROKEN STONE ROADS WITH BITUMINOUS SURFACES.

By Messrs. W. H. Fulweiler, P. P. Sharples, Clifford Richardson, and Herbert Spencer.

W. H. Fulweiler,* Assoc. M. Am. Soc. C. E.—The last paragraph of the report on this subject states: Mr. Fulweiler.

"After the bituminous material is applied it should be covered immediately with the toughest grit obtainable."

The speaker disagrees absolutely with the Committee as to the necessity of the immediate covering of bituminous surface treatments with grit. The fundamental feature is to get the bituminous material into the surface of the road, and, if it is covered with stone immediately, the stone tends by surface tension to draw the bituminous material to the surface and prevent it from penetrating into the road; it tends to give a carpet effect. That is just what should be avoided. On the contrary, the Committee should state that as much time as practicable should be allowed for the bituminous material to penetrate into the road surface.

P. P. Sharples,† Esq.—Mr. Fulweiler has opened one phase of the subject. It must be remembered, on the other hand, that if these bituminous surface coatings are applied to a macadam road which is finished in such a way that the bituminous material runs down into the body of the road and into the large stone, disintegration of the road might be brought about through the lubrication of the stone. Enough fine mineral material to absorb all the bituminous material should always be present. For the same reason, macadam roads which are built with the intention of giving them a surface coat of bitumen, should always be carefully filled with stone dust, fine gravel, or suitable sand at the time of construction. Mr. Sharples.

Clifford Richardson,‡ M. Am. Soc. C. E.—The treatment would depend entirely on the type of bituminous material in use. In the case of material like coal-tar, one would naturally wish to apply the binder at once, but, in the case of a heavy asphalt, it should be allowed to penetrate as far as it will. Mr. Richardson.

Herbert Spencer,‡ Assoc. M. Am. Soc. C. E.—The generally accepted definition of a bituminous surface is, "A surface consisting of a superficial coat, or coats, of bituminous material, with or without Mr. Spencer.

* Wallingford, Pa.

† Mgr., General Tarvia Dept., The Barrett Co., New York City.

‡ New York City.

Mr. Spencer. the addition of stone, slag, gravel, sand, or similar material"; and, in this classification, there is naturally included the type of bituminous surface made by the application of a heavy, heated asphalt on an existing macadam or gravel road.

In discussing the subject of broken-stone roads with bituminous surfaces, the Committee recommends (p. 1624), "that bituminous material of such consistency that it can be applied at a temperature below 52° cent. (125° Fahr.) is preferable to heavier material, and that the application of a quantity in excess of ½ gal. per sq. yd. is inadvisable." The speaker disagrees with this statement, and refers to the large quantity of work which has been done in various sections of the country by the use of the heavy material on a properly swept macadam road, followed by a coat of screenings or grits. Such work has been done for more than 7 years, and seems to give entire satisfaction where a mat of heavy asphaltic material can be made to adhere to the surface of a water-bound macadam road.

With the introduction of the modern automobile pressure-distributor, it has been demonstrated that it is entirely feasible to apply this material in quantities and at practically any temperature up to 300 degrees. Many miles of roads in New York, Massachusetts, and New Jersey, have been treated in this way, and the indications seem to point to a continuation of this method of treating roads.

The question of a cut-back asphalt, as compared with an asphaltic oil, is very often governed by price considerations, together with the desire, in some cases, to assist the spreading of an asphalt by fluxing it with a more volatile constituent. With the use of automobile machines capable of unloading the heaviest kinds of asphalt for work of this class, the necessity of using a cut-back product is questionable, and, in the speaker's opinion, will not secure as satisfactory results as the use of a heavier asphalt applied on the road surface at a temperature close to 300°, and then covering it with a sufficient quantity of suitable stone, screenings, or grits.

From the standpoint of cost, the heavier asphaltic material is superior to a lighter road-oil applied each season, and sometimes twice in one season. When properly applied, the heavier material should last for 4 years; and it not only protects the stone surface from the disintegrating effects of automobile travel, but presents a more uniform and attractive appearance on country and town highways.

## (5) BITUMINOUS MACADAM PAVEMENTS.

By Messrs. Clifford Richardson, W. H. Fulweiler, P. P. Sharples, J. A. Johnston, H. E. Breed, C. E. Mickey, Herbert Spencer, Prévost Hubbard, T. M. Ripley, and C. R. Allen, Jr.

Clifford Richardson,* M. Am. Soc. C. E.—Under this heading it is recommended that the bitumen should be applied after thorough rolling. Though this may be true with residuals which become very thin in a melted condition, it has been found to be very bad practice with the natural asphalts, which are much more viscous. In the use of material of this class experience has shown that the stone should be placed loosely, and not compacted until the asphalt has been distributed on it in the proper quantity, as otherwise insufficient penetration will be obtained. Mr. Richardson.

In the fourth paragraph under this heading it is provided that:

"The quantity of bituminous material used should be just sufficient to penetrate through the upper course and fill the voids; such penetration and filling is accomplished by the application of approximately 1 gal. of bituminous material to the square yard for each inch in thickness of the upper course."

Of course 1 gal. of bitumen per square yard will not fill the voids in an inch of broken stone. Neither would it be desirable, if it would do so.

In the next paragraph it is stated that the use of a pressure distributor in applying the bituminous material is essential. "Advisable" would be a better word to use than "essential", as excellent roads, when the operation is carefully done, have been constructed by hand-pouring, examples of which, of considerable age, are found in Massachusetts.

The rule adopted by Mr. Pillsbury, of Massachusetts, is a gallon of bituminous materials to each inch of surface, and certainly the results obtained speak more loudly than anything the speaker can say about it.

With such materials as are used in the construction of bituminous macadam roads, there should be some differentiation between them. It is undoubtedly true that, with the tars, what Messrs. Fulweiler and Johnston have stated may be true, but it is not true with respect to certain other materials.

W. H. Fulweiler,† Assoc. M. Am. Soc. C. E.—Attention is called to the fourth paragraph, referring to the quantity of bituminous Mr. Fulweiler.

* New York City.
† Wallingford, Pa.

Mr. Fulweiler. material used. It would seem that the Committee has suggested an excessive quantity. The speaker's experience would indicate that it is about twice too much for the first course, and, with a very carefully prepared and graded stone top, it would be nearly three times too much, successful roads having been built with only 1 gal. per sq. yd. of surface, where the surface was roughly from 2 to $2\frac{1}{2}$ in. thick. For such a surface surely not more than $1\frac{1}{2}$ gal. of some kinds of materials would be necessary. It may be that certain materials would require this large quantity, but, as a general statement, the Committee's figure is entirely too high.

In the last paragraph the Committee would have done well to call attention to the effect of stone of the softer qualities as affecting the size of stone to be used in the surface treatment. Where a rock is hard, like trap, it is undoubtedly an advantage to use the smaller size, but where limestone or other soft stone is the only kind available, a stone of rather large size should be used, in order that the average size of the particles in the finished surface, after having been crushed by the traffic, will not be smaller than, say, $\frac{1}{4}$ in., and, in addition, it should not contain an excessive quantity of dust.

The speaker's experience has been that successful results are obtained generally with about $\frac{1}{2}$ gal. on the first pouring per inch of loose stone. In other words, he would spread the upper wearing surface, say, 3 in. thick, loose, and penetrate that with $1\frac{1}{2}$ gal. per sq. yd. That is equivalent to $\frac{1}{2}$ gal. per inch of depth. For average construction, the use of 1 gal. per inch would certainly flood the road; it would produce a surface entirely too rich in bitumen, and it would undoubtedly push under traffic.

The result of digging into a number of roads that have been constructed in this way shows that the finished surface will be about $2\frac{1}{2}$ in., when constructed of 3 in. of loose broken stone.

Again, the variation in the quantity used for the flush coat is almost exclusively on the lower side, rather than on the upper side. In other words, it would be nearer to $\frac{4}{10}$ or $\frac{1}{3}$ gal. than to $\frac{6}{10}$ or $\frac{3}{4}$ gal. In the speaker's opinion, more roads have proved failures through the use of an excess than through a deficiency of bituminous material.

The New York State specifications call for $1\frac{3}{4}$ gal. per sq. yd., with a compacted top course 3 in. thick. That is about $\frac{1}{2}$ gal. per inch of loose surface. That, also, is for stone of one size. There is one point brought up by Mr. Sharples, however, that would apparently increase these figures a little. The speaker's reading of the Committee's specification was that this was the first pouring. That is what is called the penetration course. That would make a slight change, adding about $\frac{2}{10}$ gal.

The question of a specification for building a bituminous macadam pavement is a very large one, and the Committee would have done well if it had drawn attention to some of the very important factors affecting it. For instance, the type, if it may be thus termed, of stone bed that gives the best results with asphalts is radically different from that which gives the best results with tars. Mr. Fulweiler.

Therefore, the speaker thinks that the Committee might have called attention to the fact that the methods for preparing the stone before making the first pouring will have to be modified, not only in accordance with the class of stone, as to its size, but also in accordance with the class of bituminous materials to be used, and that the quantity of bituminous material will have to be varied, to take care of those features.

With regard to definiteness of statements regarding quantities, the most definite method would be to prescribe between certain limits the quantity of material to be poured per inch of loose stone on the first pouring, and then prescribe the quantity of material within certain limits to be used in the flush coat or final coat. In that way a perfectly definite quantity of material can be ascertained.

P. P. Sharples,* Esq.—The quantity of bituminous material to be used is the crucial point in the Committee's specification. The speaker is inclined to agree with the report, as his experience in Massachusetts—where he has had perhaps more than in any other one place—has been very much in favor of 1 gal. per sq. yd. per in. of compacted stone. The difference between compacted and uncompacted stone is perhaps confusing. With compacted stone, 1 gal. per in. is about right, if the seal coat is included, but, if it is not included, it is too much. It may be slightly excessive, but an examination of the specifications of a number of experienced State highway departments will show that the quantity of bitumen is in accord with the proposed specification. The Massachusetts, Ohio, and New York specifications call for, approximately, 1 gal. per in. of compacted stone, and their experience in the construction of this type, with many different kinds of bituminous materials, has been satisfactory. Mr. Sharples.

In determining the quantity of bitumen, the ultimate character of the surface should also be considered. If it is the intention to leave a surface on which the bituminous material flushes to the top, a greater quantity of bituminous material should be specified than if the surface is to be kept up by the application of surface treatments, beginning very soon after finishing the pavement.

The surface treatments, when begun within a short time after the completion of the pavement—a practice which has become common in certain parts of the country—allow a reduction of the quantity of bituminous material put into the pavement itself.

* New York City.

Mr. Johnston. J. A. JOHNSTON,* M. AM. SOC. C. E.—Many miles of road have been built in Massachusetts with 1 gal. of tar per sq. yd. Those roads were not treated with any additional seal coat, but many of them, even those built with some of Mr. Sharples' material, were used for more than 3 years before they received any further surface treatment, and that treatment was only about ⅓ gal. per sq. yd.

The speaker agrees with Mr. Fulweiler that more roads have been spoiled by excessive bitumen than by too little. In the use of bitumen, a little may be good, enough is excellent, but too much is a calamity.

Mr. Breed. H. E. BREED,† M. AM. SOC. C. E.—The specification generally used in New York State requires 2½ gal. as a total. In many instances this has been found to be too much, and in order to get a road that will stand under traffic, and will resist pushing, it has been necessary, with stone of certain qualities, to reduce the quantities of bituminous material.

Mr. Mickey. C. E. MICKEY,‡ ASSOC. AM. SOC. C. E.—Kansas City has been conducting a number of experiments on bituminous macadam pavements, and it is possible that the Committee may get some very valuable data on this subject from the authorities of that city. Experience there confirms everything that Mr. Fulweiler and Mr. Johnson have stated. A little is good, the right quantity is excellent, but too much is worse than nothing. The pavements which contain too much—they are called the "ocean waves" at present—have been in use from 2 to 3 years, and portions of some of them have had to be entirely reconstructed in order to do away with the waving effect. The speaker thinks they are now being built with larger stone and less bitumen.

Undoubtedly, the Committee intends to conduct an investigation that will provide sufficient data in formulating specifications, and will mention a bituminous material suitable for this kind of construction. The speaker is somewhat disappointed to find that some of that material has not been included in the report. On pages 1638 and 1639, the Committee has recommended certain specifications for distillate oil and coal-tar paving oil, which would be suitable as bituminous materials for an individual pavement; but it has failed to recommend a specification for bituminous materials which would be suitable for the individual bituminous pavements.

Mr. Spencer. HERBERT SPENCER,§ ASSOC. M. AM. SOC. C. E.—The success of a bituminous macadam pavement is often due to the viscosity or penetration of the material which is used. The speaker has seen specifications which call for material with a penetration ranging from 75 to more

* Springfield, Mass.
† Albany, N. Y.
‡ Lincoln, Nebr.
§ New York City.

than 200. The penetrability of the material into the stone will govern the success of the pavement. A soft material having a penetration of 175 will penetrate the voids in the broken stone more readily than a material of 100 penetration; the harder material will more naturally reach its own melting point, due to being brought in contact with the cold stone, becoming to a certain extent congealed before it has reached the bottom of the top course. This, in the speaker's experience, has been the cause of many failures of roads. The harder material will not soak down into the road before it becomes solidified; it simply lies on top as a mat; and the top inch, or inch and a half, of stone receives a greater percentage of material per square yard than the underlying stone. For that reason, success can never be expected in bituminous macadam unless the correct penetration of the asphalt has been determined for the grade of stone to be used. Mr. Spencer.

PRÉVOST HUBBARD,* ASSOC. AM. SOC. C. E.—The speaker subscribes to what Mr. Fulweiler has said, and offers as an illustration the fact that engineers do not take these things into account when building bituminous concrete roads; that is, they specify the percentage of bituminous material within certain limits, knowing that they cannot in advance decide accurately the exact quantity which will give the best results. It is only by experiment in the first stages of the work that the exact quantity can be determined, so that in specifications the limits of quantity should be given, rather than a single definite quantity. Mr. Hubbard.

T. M. RIPLEY,† M. AM. SOC. C. E.—The speaker agrees thoroughly with Mr. Fulweiler, with reference to the use of 1 gal. per in. per sq. yd. Mr. Ripley.

On page 1624, in the next to the last line, are the words "thorough rolling," Now, as the word "thorough" is determined by the character of the material, it should be omitted, or very much modified; for instance, the words "properly rolled", might be substituted.

C. R. ALLEN, JR.,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—The sizes of screen openings recommended for stone for pavements of this class is correct for trap rock, but there are many cases where economy requires the use of inferior stone. In these cases the writer has obtained much better results by starting with larger sizes and depending on the roller to crush them somewhat. In other words, the sizes of screen openings should depend on the stone to be used, and the establishing of standard sizes should be avoided. Mr. Allen.

In the writer's opinion 1 gal. of bituminous material for each inch of macadam is in many cases excessive; he is also doubtful if the quantity required is in direct proportion to the thickness.

* Washington, D. C.
† Watertown, N. Y.
‡ Albany, N. Y.
§ Received by the Secretary, January 19th, 1917.

## (6) BITUMINOUS CONCRETE PAVEMENTS.

BY MESSRS. E. A. STEVENS, CALEB HYATT, GEORGE P. HEMSTREET, W. H. CONNELL, R. B. GAGE, PRÉVOST HUBBARD, W. H. FULWEILER, C. E. MICKEY, AND W. P. BLAIR.

Mr. Stevens. E. A. STEVENS,* M. AM. SOC. C. E. (by letter).†—The classification of bituminous concrete pavements in the report seems to relate to pavements laid in one layer and of a homogeneous mixture. It is to be regretted that specific mention is not made of pavements laid in two courses, such, for instance, as a base of Type *A* with a surface of Type *C*, say, Topeka. Such a pavement is practically an open binder course with a Topeka instead of a sheet-asphalt surface. Its cost is somewhat less than sheet-asphalt, and its probable life shorter, but it has the merit of being less slippery.

On macadam bases, there appear to be some advantages in the open binder course.

Mention should also be made of the cold-mixed method. This material has now been used for some 8 or 9 years, and its advantages and shortcomings are fairly well known. It is the easiest of the bituminous concretes to lay, and the most "fool-proof." It is exceptionally easy and cheap to repair, and affords better foothold than hot mixes of the same grading. Against this, it is urged that it is not equal to the hot mixes in weight-carrying capacity, nor in length of service, though several such surfaces laid 4 or 5 years ago, and under heavy suburban traffic, have not yet called for any repairs.

Mr. Hyatt. CALEB HYATT,‡ ASSOC. M. AM. SOC. C. E. (by letter).†—The rapidly increasing use of asphalt blocks seems to warrant the mention of a few additional facts which would be of use to engineers engaged in the preparation of plans and specifications for pavements of this type.

The application to road surfaces of a bituminous concrete pavement in the form of an asphalt block, necessitates radically different methods of construction and maintenance from those used in laying bituminous concrete in sheet form, and consequently calls for somewhat different conditions of grade, foundation crown, etc.

Maximum allowable grades of 6% for blocks laid with tight joints and of 12% for special hillside construction are recommended. The hillside construction consists of laying the blocks on the customary mortar bed with ⅜-in. strips between each row of blocks. After the

* Hoboken, N. J.

† Received by the Secretary, January 19th, 1917.

‡ New York City.

mortar bed has set, the strips are removed and the joints are grouted with a 1:2 mixture of thick cement sand grout. After the grout has attained a jelly-like consistency, each joint is carefully raked out to a depth of ½ in. Mr. Hyatt.

Asphalt block, like nearly all permanent pavements, requires a curb, and, on country roads, an edging to support the sides of the pavement. The thickness of the artificial foundation of cement concrete should be from 4½ to 7½ in., and from 4 to 12 in. when laid directly on an old macadam or telford base, depending on the local conditions of the sub-grade, and the weight, density, and speed of the traffic. The thickness of the sand cushion or binder course (in this case a mortar bed) should be from ½ to 1 in. of 1:4 cement sand mortar. The thickness of the wearing course should be from 2 to 3 in.

In specifying the crown, the grade should receive more consideration. The crown should be sufficient to carry off the water. A minimum crown can be best used on the heavy grades and a maximum crown on the lighter grades. The crown may be adjusted to suit conditions, from a maximum of ⅜ in. to the foot, to ¼ in. to the foot. This matter of variation in crown warrants the serious consideration of highway engineers.

In its recommendation for joints the Committee states that cement mortar and bituminous filler only should be used. This statement should be modified for asphalt blocks, or other malleable block surfaces. The general practice is to use fine sand for the joints in asphalt block pavements, except under special conditions. The block itself is malleable when properly manufactured, and the traffic soon seals up the joints, making the pavement practically water-proof.

The specifications for asphalt block pavement on page 1628 cover the asphalt block in a general way, but, with a specification as general as this, the product could vary beyond the limits of safety, and an inferior pavement could be obtained which would still be within the limits of the specifications. The following additions are recommended for the consideration of the Committee:

*Physical Characteristics.*—The blocks should be 2, 2½, or 3 in. thick, 5 in. wide, and 12 in. long, and any variation of more than ¼ in. in length or ⅛ in. in width or depth should be sufficient cause for rejection.

All blocks should be uniform in texture and composition, straight and true to form, free from warp, wind, defective forms, or rounded or imperfectly formed edges. The planes of the opposite sides should be parallel.

*Composition.*—The blocks consist of asphaltic cement, crushed trap rock, and mineral dust. The writer agrees with the Committee that crushed trap rock is probably the best material from which asphalt blocks have been made; nevertheless, trap rock cannot be obtained in

Mr. Hyatt. all parts of the country, and there should be some provision to allow the use of a substitute having certain definite qualities comparable with trap rock.

The mineral dust used in the manufacture of the modern asphalt block is almost always made from limestone, but the general specification calls for either limestone or Portland cement. If this Portland cement item is allowed to remain in the report, it might be used by engineers in drawing up their specifications, thereby causing an unnecessary expense and hardship to both manufacturer and purchaser, and therefore this item of Portland cement should be eliminated from the recommendation.

Asphaltic cement may be made up of a combination of natural asphalt and artificial asphalt fluxes, or it may be made up entirely of manufactured asphalt and fluxes. The specification should indicate which material is desired. It is generally known that blocks manufactured from natural asphalt properly fluxed are superior in permanency to those manufactured from artificial asphalts. It is highly important, in the manufacture of a first-class block, that the asphaltic cement be of a certain consistency. It seems well that additions be made to the specifications to insure the exact qualities of the cement to be used. Cements must be hard enough to stand up under the extreme summer temperature, and still soft enough not to be excessively brittle under low winter temperature. Different climatic conditions require that the asphaltic cement be hardened or softened to meet these conditions, and, in general, as the horse traffic diminishes, and auto traffic, with its attendant heavy loads, increases, the asphaltic cement used in the manufacture of blocks should be made harder, in order to meet the more severe conditions. To insure a cement of such quality, it would be necessary to specify some of the following: Penetration, softening point, ductility, loss on heating, and brittleness. These are all standard tests, and are available to engineers making up specifications for asphaltic cements.

Where it becomes necessary to specify another stone in place of trap, it would probably necessitate changing the specific gravity, and this should be provided for.

Another item which has not been covered is the pressure required to form an asphalt block. In order to secure a block of maximum density, thoroughly filled out, it is necessary to apply a pressure of between 5 000 and 7 000 lb. per sq. in., and without such pressure no asphalt block has ever been successfully manufactured.

It is necessary to specify the grading of the mineral aggregate, and this has been incorporated in the report.

Another test which is used to ascertain the wearing qualities is known as the "rattler test." Nearly all manufacturers use some form

of rattler, and it would seem that a standard machine should be decided on, so that uniform results can be obtained. Mr. Hyatt.

The block itself should contain from 6½ to 8% of bitumen, from 15 to 25% of mineral dust, and from 67 to 78½% of crushed stone.

*Laying.*—The Committee has specified as to the laying, and these additions might be of use for engineers wishing to specify more in detail. The modern practice in London and on the Continent seems to be to lay the pavement directly on a foundation which is finished to an exact surface. The wearing course is then laid on this foundation without the usual cushion coat. In laying asphalt block, this same condition is obtained without the great expense of bringing the heavy foundation to an exact surface. On the foundation course, which need not be absolutely true, the mortar bed (1 part cement and 4 parts sand) is placed, this is brought to a true surface, and the blocks are bedded therein. This mortar bed, which ordinarily would be from ½ to ¾ in. thick, actually forms a portion of the foundation course. In other words, the result obtained is equal to that of the best practice for heavy streets in some European countries. The following extract from the New York State specifications is a good example:

"Upon the foundation shall be spread a bed of the thickness shown upon the plans, composed of one part Portland cement and four parts sand, thoroughly mixed. This mortar bed shall be struck with a template to a true surface, exactly parallel to the top of the proposed pavement surface and the required distance (2, 2½, or 3 in.) below it. The blocks shall be laid while the mortar bed is fresh, and before it has taken its initial set. All depressions and other irregularities in the surface shall be corrected by the Contractor immediately. The blocks shall be laid by the pavers standing upon the blocks already laid, and not upon the bed of mortar. The blocks shall be laid at right angles with the line of the street, and in such a manner that all longitudinal joints shall be broken by a lap of at least four inches.

"The blocks shall be so laid as to make the lateral joints as tight as possible, consistent with keeping a good alignment of the courses across the street. When thus laid, the blocks shall be immediately covered with clean, fine sand, perfectly dry and screened (to suit the local products). This sand shall be spread over the surface and swept into the joints, and shall be allowed to remain on the pavement not less than thirty days, or for such time as the action of traffic on the street shall have thoroughly ground the sand into all the joints."

Where traffic is extremely heavy, it is well to provide some form of lug or anchor to prevent the surface movement of the pavement. Several such devices are now used with success.

The writer believes that the classification of asphalt blocks under the heading "Bituminous Concrete Pavements, Class *C*", is misleading. The materials used in asphalt blocks are not identical with those used in bituminous concrete. The pavement is a special type, and should

Mr. Hyatt. be separated accordingly into a classification of its own, similar to the classification for sheet-pavements.

Although much could be added to the foregoing discussion, it seems unwise to do so at the present time, as the recommendations as outlined will insure engineers of a basis from which to form specifications which will procure the desired product.

Mr. Hemstreet. GEORGE P. HEMSTREET,* ESQ.—*Foundations.*—As the first cost of an asphalt block pavement is usually somewhat higher than the less durable types, this kind of pavement is generally used on main highways which will carry heavier and denser traffic each year. It is particularly important that a well-drained and rigid foundation be provided which is not only sufficient for present traffic, but which will carry the rapidly increasing motor truck traffic. Trucks carrying 5 and 6 tons are rapidly increasing in number, and trailers with 10 or 15 tons are not uncommon. Who can prophesy the size and speed of trucks 10 years from now? Formerly, a well-drained foundation of concrete 4 in. thick was considered sufficient; to-day, a number of pavements with 5½ or 6 in. of concrete are being constructed, and perhaps the traffic of the near future will require 8, 10, or 12 in. of concrete to absorb the shock of the heavy vehicles.

*Body Material.*—By far the largest number of blocks used have been made of trap rock, but equally good blocks can be made of copper conglomerate and a somewhat cheaper and less durable pavement can be constructed of suitable hard limestone. If limestone is used, it must be uniform, hard, and close-grained, and its physical properties should be carefully specified. Whatever kind of body material is used, it must be clean. If dirty screenings are used containing a small percentage of clay or mud, this will bake fast, when the stone is heated, and insulate the stone and the bitumen, preventing a proper bond. Grading is very important.

*Mineral Dust.*—The use of Portland cement as mineral dust is an unnecessary expense, and does not improve the quality of the pavement. Dry, pulverized, uniformly-fine limestone has proved to be the most satisfactory mineral dust.

*Asphalt Cement.*—The physical and chemical properties should be carefully specified, and these properties are not the same as those of a suitable cement for sheet-asphalt. The latter requires a soft, ductile cement to provide for contraction and expansion. Asphalt blocks need a much harder and tougher cement, and the ductility required is almost negligible, as the joints render any provision for expansion or contraction unnecessary. The character of the cement should be varied to suit the climate and the traffic conditions. In the neighborhood of New York City, or with fairly heavy traffic, a penetration of about 15

* Supt., Hastings Paving Co., Hastings-on-Hudson, N. Y.

at 77° Fahr., and about 65 at 115° Fahr., would be suitable; a hotter climate would require a penetration of from 40 to 60 at 115° Fahr. and a colder climate, a much softer asphalt cement. Mr. Hemstreet.

To specify the penetration only at 77° Fahr. is of little use, especially for pavements for hot climates; the specifications should always state the penetration at 32°, 77°, and 115° Fahr. The minimum ductility may be specified, in order to prevent the use of cheap, non-adhesive materials, but it is more satisfactory to specify the adhesion (in pounds per square inch) of the asphaltic cement when broken at freezing temperature. A minimum adhesion of 900 lb. per sq. in. is recommended. It is very necessary that the asphalt cement should not be brittle in cold weather, and this can be specified by a test for brittleness made in a Page impact machine or a modified form thereof.

*Mixing and Pressing.*—Just before mixing, the mineral ingredients should be properly heated, each one weighed out in its proper proportion, thoroughly mixed dry, the weighed quantity of asphalt cement added, and the whole mixed for at least 3 min., or preferably 4 min. The material should go to the press at a temperature of from 280° to 310°, where it should receive a pressure of at least 7 000 lb. per sq. in. Sufficient asphalt cement must be used, not only to coat every particle thoroughly, but to produce a plastic mass which will flow under pressure in the mould and produce a homogeneous block, with all edges and corners thoroughly filled. Too much asphalt cement will cause distortion of the blocks as they leave the press.

*Block Tests.*—The specific gravity of the blocks, for a known body material and grading, is an indication of the lack of voids in them, but the gravity can be increased by the admission of a percentage of larger stone, at the same time producing a poorer block.

The punch test described in the New York State specifications is valuable, if the average of a large number of penetrations is determined, because the punch may strike a large piece of stone, thus giving a false reading; scant reliance can be placed on the result of only a few tests.

The absorption test is of but little use. A block that will absorb 1% of water will be so porous that visual inspection should reject it. It is difficult to remove completely the surface water, in order to determine accurately the absorption of a fraction of 1 per cent.

The most useful test is made by a modified form of the Jones-Talbot rattler, usually known as the Hastings rattler.* About 1 sq. yd. of pavement is clamped to the inside periphery of a cylinder and subjected to the pounding and grinding action of cast-iron cubes, as the cylinder revolves. Almost any kind of bituminous pavement wears well at ordinary temperature, but suffers at extreme temperatures. For this

* Described in the *Cornell Civil Engineer*, March–April, 1915.

Mr. Hemstreet.

reason the rattler test is conducted at a low and at a high temperature. Blocks that have high rattler losses and poor appearance after test will invariably give poor results on the street, and blocks that have low losses and good appearance will usually prove satisfactory under traffic.

The blocks should be true to size and shape, and this is well covered in the New Jersey State specifications.

*Laying the Pavement.*—Usually, the blocks are laid with close joints, as described in the New York State, New Jersey, Brooklyn, Toledo, or Cleveland specifications, and when thus laid are satisfactory on grades up to 5 or 6 per cent. If the blocks are laid with $\frac{3}{8}$ or $\frac{1}{2}$-in. joints, and the joints are filled with Portland cement grout and then slightly raked, a pavement is produced which affords an ususually good foothold and is satisfactory on grades a high as 11 or 12 per cent.

Motor truck traffic produces distortion in nearly all forms of pavement, and this problem has been met by the anchor block described in the New Jersey or New York State specifications.

*Repairs.*—The speaker would suggest that the Committee specify in some detail the method of repairing cuts or street openings. It is important that the restored concrete foundation should be larger than the excavation in order to have a bearing on the shoulders. The blocks should be replaced carefully, true to grade and contour, and properly toothed in. If the cut is made late in the year, the joints should be filled with asphalt of about 140° Fahr. melting point, by the ring and ball method.

Where the pavement has become worn and uneven, it is not necessary to throw it all away, for, by taking up all the blocks and sorting out the thinnest ones, the remainder, usually from 60 to 75%, can be relaid, the joints poured with a suitable asphalt, and an additional life of from 10 to 20 years obtained.

Mr. Blair has stated that the Talbot-Jones rattler did not give satisfactory results when tested with vitrified brick, but the speaker is very sure that the lack of concordance in the results was due to the fact that the brick joints in the rattler were not tight. The bricks were put in and separated by $\frac{1}{4}$, or $\frac{1}{5}$, or $\frac{3}{8}$ in. and they were held in place by bolts passing between the bricks. With a rattler of that kind, and charged in that way, concordant results have never been obtained, but if the bolts are taken up and the bricks are placed tight together, so that they represent a smooth pavement, the same as on the street, and if those bricks are held in place with external clamps, and then the speed is regulated by a sensitive governor, the results on asphalt blocks will be within 5 per cent.

The speaker can check off a dozen rattler tests and get the same results within twenty-five points. That is done without any trouble; but it cannot be done if the blocks are separated by $\frac{3}{4}$ in., and it

cannot be done if the speed of the machine is 35 or 38 rev. per min. If it is run at 37 or $37\frac{1}{2}$ rev. per min., the results can be duplicated. Mr. Hemstreet.

The cast-iron cubes should be of approximately the same hardness, and the easiest and most practicable way to secure this is to buy them from the same foundry, and made of the same kind of iron.

W. H. CONNELL,* Assoc. M. Am. Soc. C. E.—In the report, Classes *A* and *B* are defined as follows: Mr. Connell.

"(*A*) A bituminous concrete pavement having a mineral aggregate composed of one product of a crushing plant;

"(*B*) A bituminous concrete pavement having a mineral aggregate composed of a certain number of parts by weight or volume of one product of a crushing plant and a certain number of parts by weight or volume of fine mineral matter, such as sand or stone screenings."

It is customary, of course, in all pavements of that type, to use sand similar to that used for asphalt pavements. Class *C* is:

"A bituminous concrete pavement having a predetermined mechanically graded aggregate of broken stone or gravel, either alone or combined with fine mineral matter, such as sand or broken stone screenings."

Attention is called to the fact that Class *A* is "a bituminous concrete pavement having a mineral aggregate composed of one product of a crushing plant", and in Class *B* the stone is also composed of a certain percentage of one product of a crushing plant, and in addition, a certain percentage of fine mineral matter to fill the voids.

Later in the report there is this paragraph, referring to Class *B*:

"Specifications for pavements of this class have generally stipulated that so many parts of broken stone or gravel and so many parts of sand or other fine material are to be mixed with a certain quantity of bituminous cement. By the use of this specification, unless under unusual supervision, it is not practicable to secure a well-graded aggregate. Although in many cases the mixture contains an excess of broken stone with insufficient fine material to fill the voids therein, in other cases it contains an excess of sand in which the broken stone is contained as isolated particles. In general, because of the conditions described, either bituminous concrete pavements of Class *A* or Class *C* should be used."

Confining the pavements, in the first place, to the more or less open mixture of one product of the crushing plants, and to the wording of the patent pavements—as a matter of fact, practically all these patent pavements are laid with one product of the crushing plant—this paragraph states that the Class *A* pavement is all right, and it is described as one product of a crushing plant.

* Philadelphia, Pa.

Mr. Connell.

The Class *B* pavement is described in the same way, but to it is added sufficient fine mineral matter to fill the voids, making it a much denser pavement than the Class *A*. What is advocated as a good pavement? Now, practically all bituminous concrete work is done under the specification described under Class *B*, one product of a crushing plant mixed with fine mineral matter, such as sand or stone screenings. The general specification is two-thirds of stone and one-third of sand.

That is the actual practice to-day, and the Committee's attention is called to the fact that the paragraph advising against this pavement is very misleading and contrary to successful practice. One can certainly get just as good results without unusual supervision in Class *B*, where fine material is added to one product of a crushing plant, as with Class *A*, where fine material is not added. Probably there is some misunderstanding in connection with that.

When one speaks of a pavement consisting of a mineral aggregate composed of a certain part of stone and a certain part of sand or stone screenings, reference is generally made to the Warren patents. Of course, the speaker is not discussing the Amesite at all, as that does not appear to come under either of the three classifications. The speaker thinks it would be well to take up this matter in more detail, and note the specifications for Amesite and pavements of that kind which have been successful, and are successful to-day, and include them in the next report.

Class *A* is a bituminous concrete pavement having a mineral aggregate composed of one product of a crushing plant. Nothing is stated about the mechanical grading. Class *B* is a bituminous concrete pavement having a mineral aggregate composed of a certain number of parts by weight or volume of one product of a crushing plant, to which sand is added. It is customary to add to it regular asphalt grading sand.

The stone in Class *A* is the same as in Class *B*, as it is one product of a crushing plant, the only difference being that in Class *B* fine material is added to fill the voids.

Now, in most cases where bituminous concrete pavements are laid, even where it is specified that it should have a certain percentage of $1\frac{1}{2}$-in. stone and a certain percentage of $\frac{3}{4}$-in. stone, and so on, the actual practice is to take a certain percentage of stone, passing through a 1, $1\frac{1}{2}$, or $\frac{3}{4}$-in. screen, not containing more than 5% dust, and adding to that a certain percentage of sand.

Such pavements are laid very extensively in the City of Philadelphia, and also in the counties surrounding that city. The speaker will not state that a certain pavement, in a specific locality, supposed to be mechanically graded was not mechanically graded, unless he

saw that that was done, but he thinks the general impression is that most of these pavements are not mechanically graded, and a great many of them under the speaker's supervision are not mechanically graded. The one product of the crushing plant gives them that grading, and that has come out time and again in testimony on this matter. Mr. Connell.

In the run of the crusher passing a 1½-in. ring, and not containing more than 5% of dust, one often gets a smaller percentage of voids than with the mechanical grading. That point has been fought out, and held by the Courts that the run of the crusher giving the smaller percentage of voids was an infringement on the Warren patent; and that specifying the run of the crusher is simply a subterfuge.

The speaker is not discussing patents, but calls attention to the fact that the report is inconsistent. In the first place, it states that Class *A* pavement which is the run of the crusher, is all right, and then it states that Class *B* is the run of the crusher with sufficient small material added to fill the voids. The point the speaker is trying to make is that the report should not state that Class *A* is all right with one product of a crusher plant without adding any sand, and then state that it is not good practice when sand is added to the product of the crushing plant.

The asphalt seal coat lasts longer than a tar seal coat, but in many cases just as good results can be obtained—in some cases better—and just as economical, by using a tar seal coat and renewing it every 2 or 3 years, simply with a very light application of cold water-gas or coal-gas tar over the road, and, as a matter of fact, certain kinds of roads can be kept in good condition indefinitely by treating them in this way.

R. B. GAGE,* ESQ.—Mr. Connell's statement in regard to the "run-of-the-crusher" pavement is no doubt correct. Analyses made by the speaker, representing several miles of pavements, have shown that these pavements are not what they are supposed to be, but, depending on their thickness, are composed of two or three sizes of stone; the run-of-the-crusher is never used in the Warrenite or Amiesite pavements, yet some of these pavements are laid without any quartz sand whatever. Sand, when needed, is added to the screenings in sufficient quantity to give the aggregate the desired composition. Mr. Gage.

It may be that, in some county or municipal work, the run-of-the-crusher has been used in order to avoid the cost of grading. When the stone is specified as "the run-of-the-crusher", one must take whatever the quarry happens to give, and may expect the composition to vary greatly for the same grade of stone. The stone from two trap rock quarries in New Jersey will vary in composition.

* Chemist, State Dept. of Conservation and Development, Trenton, N. J.

Mr. Gage.

It is noted that, under the heading "Mineral Aggregate, Class *A*", it is stated that great care should be taken not to burn the aggregate, yet nothing is said about burning bituminous cements. The report is not clear as to the kind of stone aggregate to which reference is made.

In regard to the different types of mixers, one of the best in use in New Jersey is the rotary drum. The product turned out by this mixer appears to be more uniform in composition and temperature than that obtained from most of the other types.

Mr. Hubbard.

Prévost Hubbard,* Assoc. Am. Soc. C. E.—Attention is called to the fact that Class *B* is the only practicable type of bituminous concrete pavement to lay, in many cases, where bituminous gravel concrete is used, and the speaker certainly thinks that an exception should be made for gravel; in other words, the Class *A* pavement, for the gravel alone, would not be satisfactory.

It is often impracticable to secure a graded fine aggregate for bituminous concrete pavements, and the speaker knows of cases where the Class *B* bituminous gravel concrete has proved satisfactory.

Mr. Fulweiler.

W. H. Fulweiler,† Assoc. M. Am. Soc. C. E.—In the paragraph on page 1626, under the heading bituminous cements, the report seems to be somewhat inconsistent. It is difficult to understand the difference between the character of a surface treatment on a broken stone road with a bituminous surface, where it is perfectly permissible to use refined tar, and a bituminous concrete pavement, where one is not permitted to use refined tar in the surface, but it may be used in the mix. The speaker thinks the point is not well taken, for the only road he ever saw where that was done was in Rhode Island, 4 or 5 years ago, and, so far as he knows, the experiment has never been repeated anywhere else. For that reason it might be well to omit all reference to it.

Mr. Mickey.

C. E. Mickey,‡ Assoc. Am. Soc. C. E.—Under Class *C*, the speaker thinks the Committee must have been in error when it included Topeka bituminous concrete pavements in its report. If the Committee had taken the trouble to get information from the places in which the Topeka specification is made for a pavement, but where the pavements are not laid in accordance with the specifications, it is thought that reference thereto would not have been made in the report.

The Topeka specification will not add any prestige to this Society, and in the final report the speaker would substitute one based on the results of actual successful experience, so that the Society could say that it has a specification that will give success.

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* Washington, D. C.

† Wallingford, Pa.

‡ Lincoln, Nebr.

W. P. Blair,* Esq.—The speaker agrees with almost everything stated by Mr. Hemstreet. His reference to the Jones-Talbot rattler, however, ought not to pass unnoticed, due to the fact that in the standardization of the rattler recommended for use in testing paving brick, there is a great variation in the character of the cast-iron blocks, and this has caused a great variation in the results attained with the rattler. If that rattler is used, it will be found to give different results in different cities, due simply to the fact that the cast-iron charges are different. Mr. Blair.

* Cleveland, Ohio.

## (7) SHEET-ASPHALT PAVEMENTS.

By Messrs. Clifford Richardson, William Goldsmith, E. W. Stern, H. L. Maier, C. E. Mickey, and J. W. Howard.

Mr. Richardson. Clifford Richardson,* M. Am. Soc. C. E.—The recommendations for a proper thickness of bituminous concrete, or close binder, for sheet-asphalt pavements, limits such thickness, when compacted, to not less than 1 in. nor more than 1½ in. A much greater major thickness is permissible, and is often desirable. In the construction of an extremely satisfactory sheet-asphalt pavement on the Victoria Embankment in London, the speaker used a thickness of 3 in. of bituminous concrete as a first course on an old water-bound macadam, on which 1½ in. of wearing surface was placed. This form of construction has demonstrated its great durability, having been in existence in that locality under the heaviest travel for 10 years, with the probability of its lasting for many years to come.

In criticizing the statement of the Committee as to the composition of a close binder, it may be said that the minimum limit for the quantity of material passing a 10-mesh sieve should be, in the opinion and in view of the experience of the speaker, made 30 per cent.

The speaker cannot agree to the statement of the Committee that the quantity of coarse sand in a sheet-asphalt surface mixture to be subjected to light traffic should preponderate over that of the finer particles. In extreme cases, a sand should not consist of more than 30% of grains retained on a screen of 40 meshes to the linear inch.†

The Committee, in taking up the question of fillers, requires that Portland cement, when used for this purpose, shall consist of at least 66% of particles passing a 200-mesh sieve. Under recent specifications of the American Society for Testing Materials, a considerably finer Portland cement is provided for.

It is stated that "The surface mixture should contain from 6 to 20% of this filler, * * *"

The speaker has never found it possible to use a quantity which would show the presence of more than 16 per cent. Attention is further called to the fact that the material or filler passing a 200-mesh screen should be really much finer than this. There are some sands, such as the loess of the West, all of which will pass this screen but would not serve as a satisfactory filler.

On page 1630, in the paragraph headed "Construction", it is stated that the components of an asphalt surface "should be thoroughly mixed by machinery until a homogeneous mixture is produced."

* New York City.

† For reasons stated on pp. 358–359 of "The Modern Asphalt Pavement", 2d ed.

The use of the word "homogeneous" is unfortunate. The Century Dictionary defines it as: Mr. Richardson.

"1. Of the same kind, essentially alike; of the same nature; consisting especially of parts of one whole.
2. Having parts of only one kind—composed of similar parts."

Of course, a surface mixture is not homogeneous, but it may be said to be, and should be, uniform.

It is further provided that:

"In cases where sheet-asphalt is constructed next to the curb, it is advisable to coat the surface for a space of 12 in. next to the curb with hot asphalt cement."

This practice was adopted in the early days of the industry, when surfaces were more porous than is the case to-day. At present it is a useless waste of material and time to provide for anything of this description.

The result of omitting the binder course would be apt to be displacement of the surface. In the early days of the industry, when the aggregate was less well balanced, and as much filler was not used, the great difficulty with sheet-asphalt pavement was that it was displaced under traffic. In order to avoid that, the binder course was developed in Washington, about 1888 or 1889. The idea of the binder course was taken from the old coal-tar pavements, and adopted in the asphalt construction. It is not absolutely necessary to have the binder course, if stability in the top course is assured, but it is a safeguard. With the advent of what is known as close binder—which is really an asphaltic concrete—the stability is in the close binder, and the thickness of the surface may be reduced to 1 in. Then there is not so much opportunity for displacement under the modern motor travel, which, as it exists to-day, is the great enemy of surfaces of all types, and especially bituminous surfaces.

With reference to asphalt block, it may be seen on the Pelham Parkway displaced into large arcs of circles, merely under the impetus of the motor cars; it may be seen likewise in sheet-asphalt, and in wood block, in the same way. At many places in New York City, where wood blocks are laid, they are displaced in the same way.

WILLIAM GOLDSMITH,* ASSOC. M. AM. SOC. C. E. (by letter).†—On page 1629, the report states: Mr. Goldsmith.

"A sheet-asphalt wearing course, consisting of predetermined graded sand, filler, and asphalt cement, should be laid to a compacted thickness of not less than 1½ in. and not more than 2 in. on a binder course of bituminous concrete consisting of broken stone or broken stone and

* New Hampton, N. Y.

† Received by the Secretary, January 19th, 1917.

Mr. Goldsmith. sand mixed with asphalt cement, the binder course having a compacted thickness of not less than 1 in. nor more than 1½ in.

In Manhattan Borough, the municipal asphalt plant used an emergency outfit to repair small holes as they appeared in asphalt pavements. A close binder was used in the hole or depression, and tamped in. It was found that many of these emergency patches lasted longer than the adjoining pavement. A modified Topeka mixture, without a binder course, was also tried and seemed to hold up as well as the regulation 1½-in. binder and 1½-in. topping. In many places, on repair work, 2 in. of close binder was used and 1 in. of topping, which also held up as well as the usual construction.

This seems to indicate that there is a grave doubt as to whether more than a 1-in. layer of topping is necessary. A 2-in. layer of close binder and a 1-in. layer of topping is a cheaper construction, and the writer believes it is just as durable.

It is suggested, therefore, that these recommendations be amended so as to include a 2-in. binder and a 1-in. topping surface.

Mr. Stern. E. W. STERN,* M. AM. SOC. C. E.—As to the shoving of the wearing surface under modern traffic conditions, the new sheet-asphalt pavement on Fifth Avenue, below 59th Street in New York City, has shoved only slightly in one spot—in front of the New York Public Library at 42d Street. This pavement is now in its third year. It seems as though the wearing surface, which has a penetration of about 30, is very suitable for modern traffic conditions. Now, if that top does not shove on the binder course, would it shove on the concrete course? Is it not worth while experimenting without the binder course?

The reason for laying the binder course, as usually explained some years ago, was that it was for the purpose of smoothing up the base, which in those days consisted mainly of old granite blocks.

The present theory that the binder prevents shoving is quite modern.

Mr. Maier. H. L. MAIER,† ASSOC. M. AM. SOC. C. E.—In Wilmington, Del., 10 years ago, there was laid a sheet-asphalt pavement, directly on a concrete base, using an asphalt cement, having a penetration of 55, and it has never shoved. In the same way 80 000 sq. yd. were laid in 1915, and about 50 000 sq. yd. in 1916. That is Wilmington's experience of 10 years. The speaker has asked many engineers: "What is the use of a binder course?" They have either avoided the question or have said something entirely irrelevant, or that "so and so recommends it."

Mr. Mickey. C. E. MICKEY,‡ ASSOC. AM. SOC. C. E.—In 1911, Dundee, a suburb of Omaha, Nebr., constructed about 50 000 sq. yd. of sheet-asphalt

* New York City.

† Wilmington, Del.

‡ Lincoln, Nebr.

without, and 12 000 sq. yd. with, the binder course. All these pavements that have been subjected to light traffic are in excellent condition to-day. A portion of one heavy-traffic street, which was laid without a binder course, has been repaired within the last year. It shoved up in waves. The penetration was about the same as that stated by Mr. Maier, namely, from 50 to 55, with the grading of New York City of 1899. The speaker thinks it would be a mistake to omit the binder course, because, in nearly all districts, it is impossible to estimate what kind of traffic pavements will have to carry within the next 5 years. In certain districts in Omaha and Lincoln, Nebr., the traffic has shifted in 6 months from light to heavy, and, therefore, at present, pavements are laid only for heavy traffic. Mr. Mickey.

J. W. HOWARD,* ESQ.—The speaker can throw some light on the subject of the binder, as he was one of the first to lay asphalt pavements with a layer of crushed stone and asphalt cement, called "binder", between the foundation and the surface layer. European cities have never used a binder, and as was the case in the earlier asphalt pavements in America, engineers recognized the necessity of having the asphalt wearing surface layer of uniform thickness, laid on a foundation finished with a uniform surface, because lack of uniformity in the thickness of the asphalt surface causes unequal subsequent compression and wear. Mr. Howard.

An asphalt wearing surface becomes softened in hot weather, and, where thick, it is displaced or shoved by traffic passing from the harder thin to the softer thick areas.

In America, where the cost of labor is high, the concrete foundations were laid with uneven surfaces, a "cushion coat" about 1 in. thick having been formerly used to even up the base. It was composed of sand and about 12% of asphalt cement. The sheet-asphalt, of sand, powdered limestone, and asphalt cement, was laid on this cushion.

About 1897, in order to reduce the cost and provide a layer which would not shove or slide, and would even up the foundations of concrete, or of old stone blocks, etc., a mixture of crushed stone and about 4% of asphalt cement, by weight, was used instead of the cushion coat and, because it bound the wearing surface to the base or foundation, it was called the "binder course" or layer.

Later, it was found that water got into this open or porous binder, which became a blanket drain under the wearing surface, causing the latter to soften from below; and the water freezing did further injury. The surface layer lasted for a time and then suddenly broke up from decay caused by the water which crept under it.

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* New York City.

Mr. Howard.

Then came the water-proof, "close binder" which Mr. Richardson has referred to, composed of fine crushed stone or gravel, the voids filled with sand, and the whole held together with asphalt cement.

Where asphalt concrete pavements are used (which pavements should be as voidless and dense as possible) it is not necessary to use binder at all, because in that case, the whole pavement surface layer is a dense binding mixture.

Relative to the density of pavement mixtures, it is the speaker's custom to test all samples of sheet-asphalt and asphalt concrete pavement mixtures, which are constantly arriving in his laboratory, for their specific gravity or density. He requires the compressed cylinder tests of density of sheet-asphalt to be at least 2.18; and those for asphalt concrete to have a density of more than 2.28. This, with trap rock and a good gradation of the whole mineral aggregate, can be made more than 2.35, and with the densest form of asphalt concrete (called Bithulithic) can be more than 2.50.

Although the gradation of the sizes of the mineral aggregate to be used is important, the proof of the results is to test the final density of the mixture by compressing it under great weight or with a heavy hammer in a suitable mould. This density test should be introduced into the specifications for all sheet-asphalt and asphalt concrete pavements.

## (8) CEMENT-CONCRETE PAVEMENTS.

By Messrs. Clifford Richardson, E. A. Stevens, W. M. Kinney, Samuel Whinery, A. N. Johnson, Walter Buehler, E. W. Stern, H. S. Mattimore, D. A. Abrams, R. A. Meeker, James W. Routh, and C. R. Allen, Jr.

Clifford Richardson,* M. Am. Soc. C. E. (by letter).†—It is stated that a thickness of from 5 to 8 in. may be considered sufficient for a concrete slab, and, if it seems advisable, for methods of economy, the thickness may be diminished from the center of the slab to the edges. Mr. Richardson.

In the writer's opinion, this is extremely bad practice. A concrete pavement should be considered as an arch, and that portion at its abutments should be thicker than at the center. The many longitudinal cracks which occur in concrete surfaces appear to the writer to be due to the fact that such surfaces have not sufficient lateral support when any settlement occurs in the sub-soil foundation. A concrete pavement or road surface should be built on the principle of bridge design, as it is really a bridge over a sub-soil.

The Committee is to be commended on emphasizing "the importance of the aggregate in making up concrete structure." The proposition is advanced, in regard to fine aggregate, that "not more than 5% should be of such fineness that it will pass a sieve having 100 meshes per lin. in."

This is, no doubt, a safe provision, but it should be regarded as not always without exception, and can only be determined by actual tests of mortar made from the sand in question.

The further suggestion that: "A denser and more uniform concrete may be made by screening the material, both fine and coarse aggregates, into different sizes and recombining" them is theoretically correct, but would hardly be possible from a practical point of view. The proposition is also reasonable to do away with the arbitrary rule of 1:2:4 or 1:3:5 in proportioning the concrete. The proportions should be regulated according to the character of the sand and aggregate available, and should also be determined by local conditions. The latter conditions may vary in different portions of the same road, calling for differently proportioned concrete or one of different thickness. It is encouraging to see that attention is being called to the determination of the proper quantity of water in use in concrete, which is too often neglected both in quantity and in uniformity.

* New York City.

† Received by the Secretary, January 19th, 1917.

Mr. Stevens.

E. A. STEVENS,* M. AM. SOC. C. E. (by letter).†—Good drainage of the sub-base for cement concrete pavements is essential. The importance of a good sand, and the proportioning of the cement to meet the peculiar sand used, and not in some arbitrary proportion, are very clearly stated, and rightly emphasized. The same is true as to quantity of water. Uniform consistency of the concrete is essential. The subject of joints can hardly be regarded as settled. It seems that prevailing practice warrants what appears to be the Committee's finding, that a joint should be used whenever work is stopped for 30 min. or more, but not oftener, though many authorities insist on joints at regular intervals.

Mr. Kinney.

W. M. KINNEY,‡ ASSOC. M. AM. SOC. C. E.—On page 1623, a certain recommended form of specification for broken stone is given. It is commonly understood that this is not to apply to broken stone used in Portland cement concrete. The report should make this clear.

On page 1631, under the heading "Materials", reference is made to the specifications for cement, devised and recommended by the Special Committee on Concrete and Reinforced Concrete. This Committee had nothing to do with the drawing of cement specifications. A Special Committee of the American Society of Civil Engineers was appointed to co-operate with other organizations in the preparation of a specification for cement, and the results of these joint labors appear in the 1916 Year Book of the American Society for Testing Materials.

The Committee specifies a maximum size of 1½ in. for coarse aggregate. There is a growing tendency to increase this to 2 in. and even larger. It is considered that the larger particles give better wearing qualities. An abrasion test for coarse aggregate could hardly be considered adaptable to pebbles and stone alike, as rounded pebbles certainly are much more satisfactory under the abrasion test than angular stone, though their wearing qualities in a pavement might be the reverse.

The Committee recommends the use of mixtures of fine and coarse aggregates giving the smallest percentage of voids, and also recommends proportioning the ingredients on the basis of the mechanical analysis rather than by arbitrary rule. It is doubtful whether a determination of the voids or the production of a mixed aggregate which would have the smallest percentage of voids will be found to be of any value whatever; in fact, the results may be very misleading, and, inasmuch as there has been devised thus far no method for using the mechanical analysis in proportioning, which gives infallible results, it is quite desirable to retain the arbitrary rule for the time being. As

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† Received by the Secretary, January 19th, 1917.

‡ Chicago, Ill.

this entire subject is being studied by a committee of the American Society for Testing Materials, and the results are likely to be rather revolutionary, it is wise to withhold recommendations until such time as these recommendations are based on facts and not on theories. Mr. Kinney.

The report states that the use of a wooden float is necessary. Although, up to the present time, this has not been thought advisable, it is now considered quite satisfactory to finish the surface of a concrete road or pavement with an 8 to 10-in. belt, operated in a manner somewhat similar to the strike board.

SAMUEL WHINERY,* M. AM. SOC. C. E.—Hydraulic concrete pavements may be said to be in about the same stage of development as sheet-asphalt pavements were 30 years ago. In the absence of sufficient knowledge, skill, and experience at that time, both engineers and contractors had differing ideas and practiced different methods as to the details necessary to secure satisfactory work. The result was that although many good sheet-asphalt pavements were built, many others were more or less defective, or were practically failures. We seem now to be passing through something like the same experience with concrete pavements. Mr. Whinery

The speaker has never lacked confidence in the ultimate success of these concrete pavements. He believes that, when properly developed by knowledge and experience, not only as to their construction, but as to their appropriate field of usefulness, they will be found eminently economical and satisfactory. The fact must not be overlooked that, as in the case of other street and road pavements, they have their limitations with reference to the weight and character of the travel to be served. Obviously, they will fail to prove satisfactory where the conditions call for a granite-block pavement.

The report of the Committee on this particular kind of road surface is comparatively brief and unsatisfactory. It contains not a word in regard to the important questions of one-course or two-course construction. Present opinion and practice differ quite widely on this matter. Perhaps, the larger area of concrete pavement thus far constructed has been laid in a single course. Future development in this matter will probably depend on whether the concrete surface is to be directly exposed to travel or is to be supplemented and protected by a carpet coat of bituminous composition. In case the first-named practice shall be found best, there is good reason to believe that two-course construction will be found to be decidedly preferable. The arguments leading to this conclusion are the same as in the case of other kinds of pavement. The two leading requisites in any pavement are strength to sustain safely the weight of passing vehicles, and hardness and toughness to resist the surface abrasion of travel. These functions are

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Mr. Whinery.

quite different from each other. Engineers have learned that a foundation course of comparatively lean concrete may be safely depended on to sustain the loads, but both theory and experience teach them that a richer, harder, and tougher material is required to resist surface abrasion. It is logical, therefore, to conclude that the best and most economical results may be secured by two-course construction, designing each course for the particular duty required of it.

If, however, the actual wearing surface subject to abrasion is to be a comparatively thin carpet course of bituminous material, then the whole body of hydraulic concrete may be designed to sustain weight only, and may be constructed in a single course of comparatively lean concrete, such as is now commonly used for pavement foundation.

Road surfaces of this latter character are of comparatively recent introduction, and time has not yet developed their true value, particularly as the bituminous carpet used has varied quite widely both in composition and consistency. In the main, however, they have proved quite satisfactory and economical, the cases of failure being generally due to materials or methods of construction, which the expert in bituminous road work would consider at least questionable.

Time and experience will be required to develop the best composition and consistency of the bituminous material for the purpose. It is safe to say that it will be neither a light road oil nor the usual asphalt paving mixture. Somewhere between these extremes will doubtless be found a composition, and a proper method of applying it, which will give the best results. It will probably be found in a heavy asphaltic oil with which is incorporated a sufficient quantity of finely pulverized mineral matter (called "dust" in the asphalt paving industry) to give it body and toughness, the mixture to be of such consistency when heated as to be readily sprayed on the concrete surface, followed by a coating of stone chips or pea gravel.

The principal difficulty heretofore encountered in the use of such a bituminous carpet has been to secure permanent adhesion between the concrete surface and the bituminous coating. The failure to secure such adhesion has generally been due to the very smooth and often "glassy" finish of a rich mortar surface. This difficulty may be largely remedied by the use of a leaner concrete, finished by tamping or rolling, instead of by floating or troweling, and by applying the bituminous composition only when the surface of the concrete is as dry as possible.

Such a bituminous surface coat, of course, will not be permanent; it will require renewal occasionally, dependent on the quantity and character of travel. From the speaker's observation and experience, he would say that, on roads and boulevards carrying a moderately heavy travel, a renewal will be required every 3 or 4 years. The first

cost of such a bituminous coating is small, and the cost per annum for renewal, which should be substantially the only cost of maintenance, would be very light. The speaker has had under constant observation a quite heavily traveled boulevard (no travel census available), paved more than 4 years ago with concrete having a carpet of coal-tar composition. The concrete was finished with a smooth and almost glassy surface, and the bituminous carpet soon separated and disintegrated over more than half the surface; but where the carpeting has adhered securely, it is still in good condition, effectually preventing any wear or abrasion of the concrete. Mr. Whinery.

Where such a bituminous surfacing is not to be used, and the concrete surface is exposed directly to the abrasion of travel, two-course construction will almost certainly prove to be the most satisfactory, and the surface course should be composed and laid with something like the same precision and care as the wearing surface of a sheet-asphalt pavement. Only the best procurable materials should be used, the determined ratios of cement, sand, stone, and water should be adhered to accurately, and the mixing should be thorough. Every effort should be made to have this surface mixture when in place as nearly uniform in composition and density as possible. Lack of uniformity in the composition and density of the wearing surface of concrete roads is by far the most frequent cause of disintegration and chuck-holes.

Regarding the proper thickness of a concrete pavement, the speaker's opinion is that, where the sub-foundation is reasonably good, the total thickness of the concrete need not exceed 6 in. Concrete is not a suitable pavement for any street where the weight of the travel will not be carried safely by 6 in. of good concrete.

The arguments in favor of making the concrete thicker in the middle than at the sides of roads of ordinary width do not seem valid, unless such a cross-section shall be found useful in preventing longitudinal cracking—of which there is as yet no satisfactory evidence. On such a road the outer wheels of vehicles follow near the outer edge of the pavement; the weight on them, due to the crowning of the road, is slightly greater; and the edge of the slab is not supported on all sides, as is the part traversed by the inner wheels. If the thickness of the concrete is to be varied at all, it would seem logical, therefore, to make it thicker at the edges than in the center.

The speaker is glad to see in the report a tendency to abandon the present practice of proportioning the materials by gross volumes. There is not time to discuss this matter at length now, but it is safe to predict that in the not very distant future engineers will fix their ratios on the basis of voids in the stone and sand rather than on volumes.

Mr. Whinery. The question of designing roads with reference to the future development of auto-trucks is attracting much attention. Vehicles of this kind, carrying such excessive wheel-loads as to be destructive to pavements and roads amply adequate for all ordinary travel are coming into use, and there is no evidence that the wheel-weight limit has yet been reached. Recent experience in New York City proves that even granite block pavements may be destroyed quickly by these excessively heavy trucks. If their continued use is to be permitted, present ideas and practice in road building must be revolutionized, and the cost of building roads must be greatly increased—perhaps doubled or even trebled. This will have to be done for the accommodation and possible slight economy of a comparatively few persons or corporations.

Roads are built for the use and benefit of the public at large, and there is no sound reason for burdening the public with the additional heavy taxation necessary to build roads and streets of adequate character and strength to carry safely the enormously heavy vehicles that a few may think will result in a slight economy to them.

Although it may not be possible or practicable to prohibit absolutely the occasional passage over the roads of excessively heavy loads, their common use may be prevented effectually by imposing license taxes that will make their use uneconomical. Briefly put, they should be taxed off the highways.

The speaker considers that this is one of the most important and urgent matters now before engineers and the road-building public, and would strongly urge that it be dealt with promptly and effectually.

Mr. Johnson. A. N. JOHNSON,* M. AM. SOC. C. E.—Mr. Richardson has mentioned the proposition to increase the thickness at the sides and make the center thin, to act as an arch. The speaker cannot see how any substantial arch action can be obtained with any possible practical construction of that sort; and, if it is theoretically proper, why not thicken the sides of the concrete base of any pavement for the same reason?

Mr. Whinery has mentioned covering concrete roads with bituminous tar as having proved satisfactory wherever it has been used. Wherever it has proved satisfactory is the exception rather than the rule, and there is no evidence of being able to maintain, beyond possibly 3 years, the bituminous covering intact. It soon loosens and scales off in large pieces, necessitating rebuilding the bituminous top, and thus adding merely a useless expense for maintenance.

The report gives the size of the coarse aggregate as 1½ in. The speaker is of the opinion, which agrees with that of many who have built a large number of concrete roads, that aggregate as large as 2 and possibly 2½ in. will prove, and is proving, desirable; and the Com-

* Chicago, Ill.

mittee on Concrete Roads of the American Concrete Institute, has recommended increasing the size of the aggregate to 2 in. Mr. Johnson.

The report states that the aggregate should not lose more than 5% by abrasion in the familiar French Deval test. A close examination of the concrete roads in service, and the character of the aggregate as shown by the abrasion test, will bring the conclusion reached by the speaker and a number of others, that the abrasion test furnishes no comparative value for a given material in use in a concrete road, and that some other form of test must be devised in order to weigh or judge between two aggregates as to their value in a concrete road under service. That is easily understood, and almost obvious. The rock is held firmly in place, and the sand mortar crust is able to take very considerable wear. The sand mortar thus acts as a sort of evener, holding up, and protecting from traffic the softer stones, and making them last longer; and, in the case of the harder stones, possibly making them wear more, by exposing them to greater wear than the softer rock protected by the sand mortar.

The proper thickness is mentioned by Mr. Whinery, he suggesting that, where it was practicable to use concrete for streets, the thickness should not exceed 6 in. The speaker would modify that by stating that the thickness should not be less than 6 in., and it should be increased toward the center to allow for some crown, making the sub-grade as flat as possible. A flat sub-grade is to be preferred to one that is crowned. A very interesting phenomenon has been observed on the California roads, which are about 4 in. thick. On these roads there has been developed a class of traffic that did not exist before. On one road, a pile of material had forced the heavy freight truck traffic to one edge, and pieces, perhaps 5 or 6 ft. long, had been broken out, with characteristic fractures. Where there was much of this heavy freight truck traffic there is evidence that a thickness of 4 or 4½ in. is not enough. Therefore the recommendations usually made for concrete roads, that they should not be less than 6 in. thick, is absolutely sound, and it is probable that they will be built thicker rather than thinner.

The report does not mention what is thought to be a very important consideration in the proper construction of concrete roads: In Sioux City there is a large mileage of concrete roads that have proved exceptional. The same is true in Macon, Ga. Apparently, the methods of construction in those places differ considerably. In Sioux City there is used a very heavy float of 2-in. plank, which requires considerable effort to manipulate in finishing the surface. In Macon, Ga., a hollow, sheet-iron roller, about 8 in. in diameter, from 6 to 8 ft. long, and attached to long poles, is rolled over the freshly laid concrete. Each of these methods performs more than the mere finishing of the surface.

Mr. Johnson.

Each compacts, and each removes the water, and this has the effect of making the concrete of the utmost density. The Macon method, developed by J. J. Gaillard, M. Am. Soc. C. E., City Engineer, is much more simple in operation, does not require as skillful workmen, and, moreover, gives a somewhat more even surface than that used in Sioux City. To secure as dense and compact a concrete as possible for a road surface is an important and essential feature.

Mr. Buehler.

WALTER BUEHLER,* M. AM. SOC. C. E.—Some 3 years ago, the United States Steel Corporation built concrete pavements of two types in its model city, near Duluth, Minn. One type had a thickness of 5¾ in. at the gutter and 8¼ in. at the crown (average 7 in.), and was reinforced; the other had a thickness of 8 in. throughout, and was not reinforced. The speaker saw these pavements a year after they had been laid, and after they had passed through one very severe winter. Cracks had developed in the unreinforced 8-in. pavement, but not in the reinforced 7-in. one, the reinforcement apparently being of more value than the additional inch in thickness.

Mr. Stern.

E. W. STERN,† M. AM. SOC. C. E.—Cement concrete pavements offer some great advantages, and the Committee is urged to gather all possible data in connection with this new and very useful type.

The speaker has examined several hundred miles of such pavements, and all which had been built properly, and in accordance with modern methods, seemed to be very good, and the maintenance cost very low. As an example, at Bay Shore, Long Island, there is a concrete road, built as a continuation of a surface treatment macadam road. The traffic conditions are exactly the same for each type. The concrete road had practically no maintenance cost in 3 years. The surface treatment macadam road cost about $1 500 per mile per year for the same period.

The speaker has seen cuts in concrete pavements repaired so well that they were indistinguishable from the adjoining surface, except for the color of the cement.

Mr. Mattimore.

H. S. MATTIMORE,‡ ASSOC. M. AM. SOC. C. E.—In regard to fine aggregate, the speaker believes that the 3% requirement on a 200-mesh sieve is superfluous, because an excessive quantity of fine material would be eliminated in the specified 5% on the 100-mesh sieve.

Mr. Abrams.

D. A. ABRAMS,* ESQ.—The statement in the report links the voids in the aggregate and density of the concrete in such a way as to indicate that in the minds of the Committee these two are synonymous. As a matter of fact, research work carried out by the speaker at the

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Structural Materials Research Laboratory, Lewis Institute, Chicago, shows that engineers have been very largely misled in believing that the voids in the dry aggregate was an important criterion as to the value of these materials in concrete. It is found that the voids in the dry aggregate is not a proper criterion. As a matter of fact, one can give examples of aggregate which will show exactly the same voids, but very different strengths in mortar or concrete. On the other hand, one can find mixtures which will give lower voids, and also lower strengths, than other mixtures with higher voids. The speaker does not refer to bituminous mixtures; but for concrete he is sure that engineers have been entirely misled by assuming that voids in the dry aggregate is a proper criterion of the relative merits of the material in concrete. Mr. Abrams.

R. A. Meeker,* M. Am. Soc. C. E.—There is a general impression throughout the country that the cure for all ills of concrete pavements is reinforcement. The speaker has seen reinforced concrete pavements which have cracked just as badly as those that were not reinforced. He has seen unreinforced concrete without a single crack in it, and reinforced concrete with cracks in which one could put his finger. This is merely one of the century-old problems of a proper foundation. All are familiar with the conditions which cause the breaking of concrete slabs in sidewalks. A concrete slab in a sidewalk, if there is soft material under one end and hard material under the other, is very likely to crack. In laying concrete for sidewalks, care is taken to remove every hard particle from under the slab, and to put in a layer of sand as a cushion to support it. On one road built in New Jersey in 1916, to obtain that result, the speaker prescribed that the sub-grade should be brought to a level 3 in. above the finished grade, that the entire sub-grade should then be plowed, and then rolled and consolidated, after which the concrete was laid. In this way an even and uniform settlement was obtained. Reinforcement alone will not prevent a concrete slab from cracking. Mr. Meeker.

James H. Routh,† Assoc. M. Am. Soc. C. E. (by letter).‡—Inasmuch as the Committee refers to the section of its report on concrete pavements for information as to the construction of concrete foundations, the writer wishes to discuss this section from the point of view of the foundation. Mr. Routh.

It is believed that a more detailed statement as to the character of the fine aggregate could be made with benefit. It is also believed that even more stress should be laid on the quantity of water to be used in preparing concrete. The writer is of the opinion that this

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Mr. Routh. quantity should be specified, as well as the quantity of other materials, as water plays a most important part in the manufacture of good concrete.

In the construction of concrete pavement foundations, there is, perhaps, more carelessness evident in the control exercised over the water than over any other factor entering into the building of a pavement. Generally speaking, this carelessness takes the form of allowing the use of excessive quantities of water. A very wet mix is easier to obtain, flows more readily from the mixer spout, and requires less labor to place than a mix of the proper consistency. That excessive quantities of water must result in segregation of materials, washing out of a portion of the cement and a consequent weakening of the entire structure, is lost sight of. Frequently, the point of view is that "anything goes so long as it is covered up."

The foundation of a bridge, a building, or a dam is considered to be at least equal in the importance to the superstructure. The writer sees no reason for not considering the foundation of a pavement also fully as important as its superstructure or wearing surface. It is his experience that at least 50% of pavement failures are due to the policy of allowing "anything to go so long as it is covered up." Therefore, he believes that the Committee may well emphasize the importance of care in constructing the foundation courses of pavements, and that, in particular, it should emphasize the need for care, and still more care, in the control of the water factor in concrete.

Another point it is desired to raise is that of sprinkling and protecting concrete during the process of hardening. A year ago the Committee stated that provision for this should be made "if possible." This statement was criticized from the viewpoint that the phraseology "if possible" should not be incorporated in specifications, and that this Society should not officially recognize any arguments as to the impossibility of contractors and builders complying with recognized good practice. It is the writer's opinion that this criticism was just; this year, however, the Committee has not only omitted the "if possible", but all mention of the matter. Good concrete is essential to good pavements. That is admitted. Then why should requirements which are necessary to secure good concrete be waived because of no real reason except the disinclination of contractors and builders to take pains with what is to be covered up?

Mr. Allen. C. R. ALLEN, JR.,* ASSOC. M. AM. SOC. C. E. (by letter).†—It is the writer's experience that, in many cases in cement-concrete pavements, stone larger than $1\frac{1}{2}$ in. may be used, and that larger sizes are in fact desirable if well graded.

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† Received by the Secretary, January 19th, 1917.

The report, in describing the construction of cement-concrete pavements, seems to be incomplete without some mention of the time devoted to mixing. The writer suggests the insertion of the following, from the *Proceedings* of the Second National Conference on Concrete Road Building: Mr. Allen.

"The mixing should be continued for at least 1 min. after all materials are in the mixer and before any of the concrete is discharged. The speed of the mixer should not exceed 16 rev. per min.; the number of revolutions per batch should not be less than 10; however, the time, and not the number of revolutions, should be the factor for determining proper mixing, in order to distribute the water through the batch."

## (9) BRICK PAVEMENTS.

By Messrs. Will P. Blair and Maurice B. Greenough, Clifford Richardson, Prévost Hubbard, J. W. Howard, James W. Routh, Frank B. Dunn, William C. Perkins, and W. M. Kinney.

Messrs. Blair and Greenough.

Will P. Blair,* Esq., and Maurice B. Greenough,* Esq. (by letter).†—On page 1632 it is stated that the maximum grade of cement-grout filled brick pavements should be limited to 6%, and that, for steeper grades, when "hillside" brick are used, "bituminous joints should always be used."

The writers submit:

1.—That, when "hillside" brick have been used on steep grades, only in rare instances has any filler but cement-grout been used, and, furthermore, that the use of "hillside" brick with cement-grout filler is superior in all respects, in service rendered, to that produced by bituminous filler with similar brick.

2.—With monolithic construction, no filler but cement-grout has ever been used.

*Artificial Foundations.*—The discussion on artificial foundations on page 1633 would imply:

(*a*) That a minimum of 4 in. for concrete foundations is the best practice.

(*b*) That dispensing with the concrete foundation hinges on low-priced brick, exceptional bearing qualities of the underlying soil, and lack of funds to provide a more substantial road surface; and that, when these conditions obtain, sand should be the joint filler.

(*c*) At the end of the first paragraph on page 1633, the statement is made: "If the roadway is not kept clean, the material which accumulates on the surface of the wearing course will protect it from injury, and its function will then be simply to provide a foundation for an earth road."

The writers submit:

1.—That 80% of the pavements of the City of Cleveland, Ohio, and also to the extent of thousands of square yards in many other cities, are laid on the natural earth foundation with cement-grout filler. The service rendered, frequently without expenditure of any money whatever for repairs or maintenance from wear and tear, covering a period of from 10 to 20 years, relieves this type of construction from any implication of ill adaptation or of being a makeshift. In a wide territory throughout the South, this type of con-

* Cleveland, Ohio.

† Received by the Secretary, January 19th, 1917.

struction has been used to so large an extent that it can scarcely be ignored. Messrs. Blair and Greenough.

2.—That thousands of square yards of the best brick pavements are laid on broken stone and old macadam foundations.

The writers are at a loss to understand why it is stated that a brick street or road, any more than one of any other type, when it becomes dirty, should be regarded as merely a foundation of an earth road. Experience does not warrant this statement.

*Cushion Course.*—On page 1633 the following statements are made:

(*a*) Recommending the sand cushion for brick pavements, presumably with cement-grout fillers as well as other fillers.

(*b*) The semi-monolithic method of laying brick pavements on a sand-cement super-foundation is referred to, but only reference thereto is made. This type is criticized on the grounds of being not well adapted to withstand shock.

(*c*) The recommendation for a bituminous cushion course implies that this is a well-known and commonly-used method of laying brick pavements.

The writers submit:

1.—That all elements of value recognized in the use of the sand-cushion type have been found to be well conserved in the green concrete foundation type, as well as increasing, adding to, and rendering certain, economies in use; by such methods, also, hazard in construction is greatly minimized, and the cost of performing the work is lessened. It is well established, also, that this type of construction possesses a flexibility in design, adjustable to the varied natural conditions due to locality, to the varied service required of a road, as well as meeting possible limitations in financing, which so often determine whether a road is or is not to be built—advantages to which the public is surely entitled.

2.—That the use of bituminous cushion courses has never passed beyond the purely experimental stage in isolated places, if even that far, and therefore reference thereto should take due account of this fact.

The writers cannot but feel a deep sense of the importance and influence which will attach to a report on brick pavement construction by a Committee of this Society, both in respect to its worth to the country and to the paving brick industry.

This dual interest in the report lies in its correctness, in order that the public may be benefited by the dissemination of knowledge of the best practice in brick road construction; and, closely associated with this thought is the benefit to the paving brick industry which arises in a public well served.

Impressed thus, as the writers are, and at the same time entertaining a most sincere belief that these suggestions will meet in the

Messrs. Blair and Greenough. largest measure the Society's position, as well as supply the public with the largest measure of its needs in this direction, they are willing, by any and every means, to substantiate in the minds of the Committee the wisdom of this position, and to that end will undertake to furnish all the evidence and testimony necessary to establish such a conclusion.

Mr. Richardson. CLIFFORD RICHARDSON,* M. AM. SOC. C. E. (by letter).†—The Committee is to be commended for bringing to the attention of engineers the bedding of brick in cement mortar instead of placing them on a sand cushion. This is a rational form of construction which should have been adopted years ago.

Mr. Hubbard. PRÉVOST HUBBARD,‡ ASSOC. AM. SOC. C. E. (by letter).†—Many conflicting opinions have been expressed by various engineers as to the relative merits of the monolithic and sand-cushion types of brick pavements, as well as cement-grout and bituminous joint fillers for brick. Though these opinions have undoubtedly been based on personal observation or experience, very little definite data have been presented on the subject, other than that included in certain generalities. It was felt that information of some value might be obtained by conducting certain physical tests on sections of brick pavements constructed in the laboratory. On the suggestion of C. H. Moorefield, Assoc. M. Am. Soc. C. E., a series of laboratory tests was therefore started a few months ago in the United States Office of Public Roads and Rural Engineering. Thus far these tests, which should only be considered as preliminary, have developed a few facts which appear to be of sufficient interest to warrant presentation.

Because it could most readily be made, and also because it gave promise of developing facts of most interest, an impact test was decided on for preliminary investigations. The large Page impact machine of the Office was made available for testing sufficiently large sections of pavement by inserting a flanged section under the main supporting column, so that the plunger could be raised high enough to place beneath it a test specimen 10 in. or more in thickness.

The sections of pavement for test were constructed in a rectangular bottomless box mould or frame made of ⅜-in. boiler plate. This frame was sufficiently deep to allow for a 6-in. concrete foundation and a 1-in. cushion in addition to the paving brick. Allowing for joints, the area of the test section was that of six bricks arranged as shown in Fig. 1.

For all tests here reported a 1:3:6 concrete base was cast in the frame and allowed to age for at least 7 days before testing. On the

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‡ Washington, D. C.

base the bricks were set in some cases on a 1-in. or 2-in. compacted Mr. Hubbard. sand cushion, and in other cases on a 1:3 cement-mortar cushion spread dry on the green concrete. Some sections were grouted with a heated bituminous filler.

Tests were made by placing the entire specimen in the frame on the anvil of the impact machine so that the plunger rested on the center of the upper surface of the center brick. The blow of a 10-kg. hammer was then delivered in increasing increments of 5 cm. drop. The rebound of the hammer was measured for each blow, and the height of that blow which caused failure was noted, together with any particular conditions of interest which developed in the specimen during test.

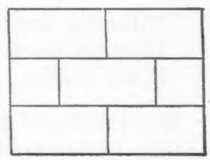

FIG. 1.

In these tests two lots of paving brick—repressed, and wire-cut lug—were obtained from a brick manufacturer, with a request that at the plant each lot be selected with a view to including as uniform brick as possible. When subjected to the rattler test, however, there was found to be considerable variation in the loss of individual bricks, although the average was low, as shown in Table 1.

TABLE 1.—TEST OF PAVING BRICKS.

| | Repressed. | Wire-cut lug. |
|---|---|---|
| RATTLER TEST: | | |
| Minimum percentage of loss | 14.2 | 13.7 |
| Maximum percentage of loss | 20.2 | 24.2 |
| Average percentage of loss | 17.2 | 17.8 |
| TOUGHNESS TEST (10-kg. hammer): | | |
| Minimum blow, in centimeters | 46 | 40 |
| Maximum blow, in centimeters | 60 | 50 |
| Average blow, in centimeters | 51 | 45 |

From Table 1 it is also evident that considerable variation exists in the resistance to impact offered by individual bricks, the toughness values given being the results obtained by testing eight individual bricks of each type.

The cement and aggregate, used in the preparation of the concrete, mortar cushion, and grout, met the usual test requirements. The asphalt filler used in these tests showed the following characteristics:

Mr. Hubbard.

ASPHALT FILLER.

Specific gravity, 25°/25° cent........................ 1.002
Penetration, 25° cent., 100 grams, 5 sec................33
" 0° cent., 200 grams, 1 min...............26
" 46° cent., 50 grams, 5 sec................50
Loss, 163° cent., 5 hours.............................. 0.1%
Penetration residue, 25° cent., 100 grams, 5 sec......30
Soluble in carbon disulphide.......................... 99.95%
Organic matter insoluble.............................. 0.04%
Inorganic matter insoluble............................ 0.01%

100.00%

Bitumen insoluble in 86° B. naphtha................... 32.38%
Bitumen insoluble in carbon tetrachloride............. 0.75%
Fixed carbon.......................................... 15.4%

The results of tests on sections of pavement constructed with the repressed brick are shown in Table 2.

TABLE 2.—TESTS OF SECTIONS OF PAVEMENTS.

| Filler | Cement Grout. | | | | | | Bituminous. |
|---|---|---|---|---|---|---|---|
| Cushion | 1 in. Mortar. | | | 2 in. Sand. | | 1 in. Sand. | 1 in. Sand. |
| Age of grout, in days | 7 | 14 | 28 | 7 | 28 | 28 | 2 |
| Maximum blow, in centimeters | 80 | 90 | 95 | 80 | 95 | 80 | 90 |
| Grout cracked at, centimeters | 70 | ? | 80 | 40 | 40 | 40 | No |
| Maximum rebound, in centimeters | 23 | 26 | 25 | 7 | 10 | 8 | 5 |
| Base cracked | Yes | Yes | Yes | No | No | No | No |
| Brick left base | Yes | Yes | Yes | ...... | ...... | ...... | ...... |

Check tests were made in a number of cases and found to run close together. Where such checks were made, the average results are given in Table 2.

On comparing these results, it will be seen that the following facts are indicated, if not conclusively proved:

1.—As compared with a sand cushion, the monolithic type of construction does not protect the brick from the effect of impact, but does tend to increase the resistance to impact of the grout filler when the blow is delivered on the brick. In the monolithic structure, therefore, there is less tendency for the grout filler to separate from the brick, although failure of the grout may be expected to occur before a general failure of the brick.

2.—In the monolithic structure, failure of the brick under impact resulted in failure of the entire structure, including the foundation,

but, with a sand cushion, no failure in the foundation occurred under the impact which caused failure of the brick. This fact seems to throw into question the recommendation frequently made of using a thinner foundation for the construction of the so-called monolithic type. Mr. Hubbard.

3.—The capacity of the monolithic pavement for absorbing the shocks of impact is much less than where a sand cushion is used.

4.—There seems to be no great difference in effect between a 1-in. and 2-in. sand cushion. When the grout failed, however, there was a greater vertical displacement of the center brick on the 2-in. than on the 1-in. cushion.

5.—The bituminous filler protects the brick from the effect of impact fully as much as the cement grout filler, and does not itself fail, even when the brick fails under impact. It should also be noted that no apparent vertical displacement of the brick occurred where the bituminous filler was used.

6.—As compared with cement grout, the bituminous filler apparently increases the capacity of the sand cushion pavement to absorb the shock of impact.

7.—In comparison with the results of impact tests on individual bricks, it is of interest to note that in the pavement this resistance is almost doubled, irrespective of the type of filler.

A few tests have been made on wire-cut lug block sections of pavement, with results quite similar to those given herein. It is proposed to continue this investigation with various makes of brick, and to use other types of filler and cushion, such as bituminous grout or mortar.

J. W. Howard,* Esq.—The mixing of a little Portland cement in the sand in all pavement cushions is excellent, because it prevents the sand under brick, stone, or wood paving blocks from being displaced and washed away toward the gutters. Mr. Howard.

James W. Routh,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer questions the statement of the Committee, that the function of the sand cushion is to give resilience to the wearing surface. Recent practice has been to sprinkle and roll the sand cushion prior to laying brick. If there is anything much less resilient than damp and thoroughly compacted sand, the writer would like to know about it. Mr. Routh.

In considering the question of construction, no mention is made of the proper method of preparing the cushion course, except that great care should be taken to avoid disturbing the surface of the cushion after it has been brought to true grade by the use of a template. It is believed that more failures of brick pavement are due to the sand

* New York City.

† Rochester, N. Y.

‡ Received by the Secretary, January 19th, 1917.

Mr. Routh. cushion than to almost any other factor. Unless the cushion is thoroughly compacted and well confined, the sand will shift. This is particularly dangerous in the case of pavements with grouted joints. The writer believes that, if a sand cushion is used at all, care should be taken to have it thoroughly rolled and compacted before the bricks are laid. He is also of the opinion that better brick pavements would result if the use of the sand cushion were abandoned. A cement-sand cushion, or a mortar cushion, appears to give much better results.

Mr. Dunn. FRANK B. DUNN,* ESQ. (by letter).†—The engineers of the writer's company have spent years in studying and experimenting with brick pavement construction, and at great cost. The results have been contrary to some of the important suggestions made in the report.

The use of a sand cushion is no longer regarded as good practice. Monolithic and semi-monolithic construction have so completely demonstrated their superiority to the sand-cushion type that the latter is being abandoned all over the country, and in a very few years it will disappear entirely.

Contrary to the statement on page 1617, a grouted brick pavement, of either the cement-sand or green-concrete foundation type, is practically noiseless. This fact is no longer disputed by engineers who have observed such pavements. These pavements are less noisy than bituminous filled pavements with open joints, as they allow the brick to cobble and become rough.

The suggestion that a bituminous filler be used on brick pavements where the grade is in excess of 6% is predicated on the theory that the soft filler allows the brick to cobble and the pavement to become rough so that horses may obtain a foothold. The theory is correct, but it holds as well on level pavements where cobbling of brick is facilitated by bituminous fillers.

Experience has demonstrated, also, that a special hillside brick, especially of the wire-cut lug type, makes a much better and stronger pavement if bonded with cement-grout filler, as such a pavement can be used on grades of 18% and give horses good foothold, yet not increase traction resistance above that offered by a smooth brick pavement. The last mentioned fact is due to the circumstance that the short, broken grooves, running across the wearing surfaces of the bricks, afford a smooth wheeling surface—the wheels of vehicles bridging the longitudinal joints and finding an even bearing on the plane surfaces on alternate brick.

The suggestion that a bituminous filler may be preferred to a cement-grout filler, on account of lower cost of street-opening repairs, does not seem to be one to which the Society can afford to commit itself.

* President, The Dunn Wire-Cut Lug Brick Co., Conneaut, Ohio.

† Received by the Secretary, January 19th, 1917.

It is customary to construct brick pavements with a view of durability, not for the purpose of tearing them up. Municipalities and rural communities want the best type of construction, one that will give the longest service. The writer's experience has demonstrated that a cement-grout brick pavement gives the longest and most satisfactory service, remaining smooth and even for years, if properly constructed. Mr. Dunn.

The writer submits for consideration the fact that, in the matter of brick pavements, experience and observation, covering a long period of years, lead to conclusions the reverse of some of those embodied in the report of the Committee.

This is not a matter that concerns manufacturers of materials; it is a practical engineering question involving proper constructional methods, and the writer expresses the conviction that if the Society commits itself to the suggestions in the report to which attention has been called, it will in some points endorse methods rapidly being abandoned, and in other matters it will commit itself to constructional methods which practical paving engineers generally condemn.

WILLIAM W. C. PERKINS,* M. AM. SOC. C. E. (by letter).†—Owing to the importance of brick pavements and their proper construction, the writer believes the report should be more comprehensive. There has been a great advancement in the manufacture of paving bricks during the past 5 years, and also, as engineers believe, in the proper construction of brick roads and pavements. The manufacture of such bricks is one of the large industries of the United States, and many millions are laid annually. Mr. Perkins.

The writer is aware that this is only a progress report, and feels assured that the final report will be more complete and describe more in detail the new types of brick paving and their proper construction.

In the matter of grades, the writer does not believe the Committee should establish any arbitrary rule; the custom has been to use a special "hillside" brick on all grades greater than 6%, and many pavements are laid with special brick, cement-grouted, on grades as high as 10 and even 14 per cent. Some engineers generally require brick of "hillside" type on roads built of other materials, whenever the grades are greater than 6 per cent.

The suggestion that a bituminous filler be used on brick roads when the grades are greater than 6%, and the assertion on page 1632 that "with special 'hillside' brick, bituminous joints should always be used" seem to the writer to be rather positive statements and contrary to present practice. The merits or demerits of the various fillers were discussed to some extent at the meeting in 1916, and the report of the Committee at that time (if the writer's memory is correct) was in favor of cement grout for brick roads and pavements.

* Conneaut, Ohio.

† Received by the Secretary, January 19th, 1917.

Mr. Perkins. The United States Department of Agriculture* has described the various types of fillers, and has recommended cement-grout "as better adapted for filling the joints of brick pavements than any other material which has been commonly used for that purpose."

As the tendency of a bituminous filler is to run out of the joints during warm weather, one can imagine what would happen at the bottom of a hill on grades as steep as 6% if a filler of this type were used. Again, if used with a regular paving brick, the roughness to be given to the pavement is based on the theory that the bituminous filler will come out (as it does) and allow the brick to "cobble." This theory is correct, and it holds good on level pavements, where "cobbling" of brick is facilitated by a bituminous filler. Experience has shown that a cement-grout filler should be used generally on brick pavement, regardless of grades.

With reference to artificial foundations, the writer believes the Committee should not adopt a minimum depth of 4 in., but, during the coming year, should investigate types of foundations of less than 4 in. In several places in Illinois, and also in the South, bricks are being laid monolithically on foundations of less depth than 4 in., and thus far have given good service. The writer believes it is the duty of an engineer to design for economy as well as for durability, and that the proper depth of a foundation depends largely on local conditions, the nature of the sub-soil, and the volume and weight of the traffic. This is a fertile field for investigation by the Committee.

It is a mistake to use a sand filler for a temporary improvement. There are several very serious objections to sand as a filler; it does not prevent water from penetrating through the joints to the foundation; it does not bind the units of brick together, so as to enable them to resist traffic; it is easily worked out in cleaning the pavement, or by rain; and, as it does not protect the edges of the brick, in a short time the pavement becomes rough and uneven and the bricks chipped and cobbled. It is better to build well at the start, or adopt some type of pavement which can be used eventually as a foundation for brick.

In reference to the "cushion course" the writer must confess surprise at the stand the Committee has taken. Many engineers believe that the only function of the sand cushion is to provide a uniform bearing for the brick wearing surface, the sand taking care of any irregularities in the depth of the brick or in the surface of the foundation. The writer questions any advantages gained in trying to give resiliency to the wearing surface with a cushion of sand. Better results are obtainable by endeavoring to unite the wearing surface with the foundation and then depending on the sub-soil to give any needed resiliency.

* *Bulletin No. 373*, on Brick Roads, issued by the Office of Public Roads in 1916, pp. 17 and 18.

The advantages of the monolithic and semi-monolithic construction, and the disadvantages of the cushion of sand have been examined so fully in the technical press during the past 2 years that the writer need not discuss them herein. He will merely state that experience has proved all that is claimed for this type of construction, and that the laying of the brick directly in the green concrete (for highway uses), and the use of a bed of cement-sand 1 in. in depth (which is considered as 1 in. of foundation for sub-soil) has come into general use and is recognized as the modern method of brick paving. Any bituminous bed or cushion is antagonistic to the fundamental idea of this type of construction, which is a bonding or cementing together of the integral parts of the pavement—the brick, bed, and foundation—into a rigid beam or monolith. Mr. Perkins.

In reference to materials, the writer thinks it would be of great value to paving engineers if the Committee, during the coming year, would investigate the results obtained in the standard rattler with the large shots removed. It is believed that the so-called variations in the tests are due to the accidental breaking of good brick by the large shot, and that, if these were omitted and only small shot used, the results would be more representative of the wearing qualities of the brick.

As the standard rattler was developed by the American Society for Testing Materials, it does not seem to be necessary to name any brick association in describing the test for brick.

The absorption test is not generally considered, engineers believing that the test for abrasion fully determines the wearing value of a brick.

The writer trusts that the Committee in its final report will describe in detail the size, type, and general appearance of the bricks which it considers should be used. Several types of paving bricks are being made, and the manufacturers would welcome a visit from the Committee, and would gladly demonstrate the care taken in making them. The size and nature of the lugs or projections on the sides of the bricks should be described, and every effort should be made to have these lugs of such uniformity and of such a character that the bonding material will flow freely to the bottom of the brick.

During the past year, the depth of the brick, especially for monolithic construction, has been reduced to 3 in., in some instances, thus making a saving in transportation charges. The claim is made that a 3-in. brick on 5 in. of concrete will give as durable a pavement as a 4-in. brick on 4 in. of concrete, provided both are laid monolithically. This is another field for investigation.

In regard to construction, the durability of a brick wearing surface depends on three essentials: good design, good material, and good construction. Therefore, the writer believes that all operations should be

Mr. Perkins. described carefully, from the preparing of the sub-grade to the opening of the street to traffic. In the construction of the brick wearing surface, details should be given as to preparing the cement-sand bed, the laying, rolling, and wetting down of the brick, and the proper preparation and application of the filler. Many pavements of all types have failed because of ignorance of good construction.

The engineers connected with the writer's company have spent years in the construction of brick pavements, in studying their defects, and in experimenting, and the results of their studies and investigations are at the command of the Committee.

Mr. Kinney. W. M. KINNEY,* ASSOC. M. AM. SOC. C. E.—Under the heading, "Brick and Slag Block Pavements", the following statement appears, "With special 'hillside' brick, bituminous joints should always be used." It is difficult to understand why the Committee made this recommendation, in view of the successful use of cement grout under such conditions, and the rather rare use of bituminous joints for hillside brick. It would also be interesting to know of cases where a mixture of sand, or stone chips, and a bituminous cement has been used as a cushion course under brick pavements, on the successful use of which the Committee recommends this and similar construction for stone block pavements. The cases where such construction has been used are certainly rare, and it seems to be quite unusual to discredit types of construction which are in successful use, substituting therefor a type which has been used, if at all, only rarely.

* Chicago, Ill.

## (10) STONE BLOCK PAVEMENTS.

By Messrs. Edward Whitwell, C. D. Pollock, H. W. Durham, J. W. Howard, W. H. Connell, E. W. Stern, and W. M. Kinney.

Edward Whitwell,* Esq. (by letter).†—In no case is sandstone considered either suitable or economical, except for rubble foundation purposes, a use which does not appear to be general in America. Mr. Whitwell.

The writer is inclined to disagree with the recommendations relative to the size of stone blocks for paving, as long experience tends to show that, the greater the size of the block, the more pronouncedly uneven becomes the surface as wear takes place. The sizes now in general use on the European continent approach more closely to those given for the Durax pavement referred to in the report, than those given by the Committee; and the trend of opinion inclines to sizes which almost coincide with those of Durax. Of course, it must be noted that though Durax has introduced a much more acceptable form of paving, chiefly due to the reduction in the size of the blocks, and though its long life probably has some economical advantage, yet any form of stone block paving, by reason of its extreme noisiness, its tendency to uncleanliness, and its first high cost, can never become a popular or recommendable form of surfacing. The reduction in the size of the blocks in all such surfacing, though a great improvement on the old forms, has only tended to reduce the number of disadvantages such pavements possess, and in very few instances is a return to this ancient system advisable.

C. D. Pollock,‡ M. Am. Soc. C. E.—Near the top of page 1635, the crushing strength of stone blocks is specified as not less than 20 000 lb. per sq. in. This seems to be a step backward; for engineers have been discarding this test, as not being a reliable index to the quality of stone for paving material. There are many cases where a comparatively soft stone has given ideal results as a paving material, and there are cases where stone having a high crushing strength has given poor results. The main thing in stone block pavement is to have blocks of uniform quality, not a mixture of very hard and very soft stones in the same pavement, as they will not wear evenly. The crushing strength test has been very generally discarded. Some engineers still have it in their specifications, but even these do not place much dependence in this test. Mr. Pollock.

* London, England.

† Received by the Secretary, January 19th, 1917.

‡ New York City.

Mr. Durham. H. W. DURHAM,* M. AM. SOC. C. E.—The speaker's recent experience with granite block on suburban road pavements, has served to strengthen his belief that it is unnecessary to specify a crushing test. It would be very desirable to conduct extensive abrasion and other tests on all kinds of stone that have been adopted, before stating definitely the exact limits in hardness. In connection with granite pavements, a much more important feature than specifying arbitrary tests is the question of inspection. Of course, it is necessary to have certain specifications in purchasing blocks, but Mr. Pollock has brought out very clearly some of the points that should be considered, and the fact that the crushing test is not the criterion.

Mr. Howard. J. W. HOWARD,* ESQ.—In regard to the 20 000-lb. test, it is quite certain that all hard granite blocks would stand that test. Where New Hampshire granite or softer granites are being laid with cement grouted joints, it is not necessary to have so high a crushing test. For such use, a test of 16 000 lb. is sufficient.

Mr. Connell. W. H. CONNELL,† ASSOC. M. AM. SOC. C. E.—The speaker thinks the 20 000-lb. crushing test is more or less obsolete, and was based on specifications that were only intended to admit certain granites from certain quarries. However, there should be some sort of test. The speaker has made a number of tests of granites from New Hampshire and from the South; they stand about 17 000 lb. If granite is much softer than that, it is doubtful whether it will give very satisfactory results. These remarks are offered simply as a suggestion to the Committee—that there is need of some kind of a test for granite. One cannot depend entirely on stone coming from a certain quarry, because there are many cases where engineers are not familiar with the different quarries throughout the country.

Mr. Stern. E. W. STERN,* M. AM. SOC. C. E.—As to the adoption of a 20 000-lb. minimum for the crushing strength of granite, the speaker has doubts.

He would suggest to the Committee some investigations relative to the depths of paving blocks. Chicago is using a 4-in. block, and New York City uses 5-in. Perhaps, if 4-in. blocks were used in New York City they would lie a little bit closer, and give a closer and better joint.

In connection with the sand bed, the speaker would not use the word "cushion." The sand bed, if it acts as a proper bed, is not a cushion, because the sand should not flow away; where the sand had been held in place as a bed for a number of years, it becomes almost as hard as mortar. The objection to sand as a bed is that it does not always stay in place, that leakages of water wash it away, particularly

* New York City.

† Philadelphia, Pa.

where cuts are made in the pavement, and, if the slightest compression ensues, a large area outside of the cut is sometimes affected. New York City, therefore, for the last 2 years, has used under granite a mortar bed of 1 part cement and 3 parts sand, with very good results. This bed is mixed practically dry, and slightly sprinkled; the blocks are rammed in the usual way. As to resiliency, with rubber-tired traffic there is no object in trying to obtain any resiliency in the bed. With steel-tired traffic it is doubtful whether the resiliency given by a bituminous bed would be worth while. Mr. Stern.

In streets where granite pavements are possible, the cement grout filler is the best; this is the experience in New York City, and it coincides with that of Worcester and other cities which have tried it. The question of cuts, however, as the Committee properly says, is paramount in New York City. In the Borough of Manhattan, in 1916, there were 25 000 cuts, and 35 000 in Brooklyn. In a very short time a cement filler would be gone, and, unless it is cured properly, it is no better than bituminous filler, and simply keeps traffic off the street for several days; whereas bituminous filler permits it to go on almost immediately.

W. M. Kinney,* Assoc. M. Am. Soc. C. E.—On page 1636, under the paragraph, "Construction", the Committee has argued at considerable length against the use of cement grout fillers. It does not seem to the speaker that the fact that a pavement can be easily removed and restored is an argument for that particular type, but rather one against it. If engineers make the taking up and restoration of pavements more difficult, the work which necessitates such action will be done more frequently prior to the construction of the pavement. As such cost is borne by the party having the cut made, what difference does it make to a city, provided the best pavement is secured by using a cement grout filler? Those who take up and restore the pavement should be required to put it back in the condition in which it was found. The fact that many cities are removing block pavements previously put down with bituminous filler, and re-laying these pavements with cement grout filler, indicates the satisfaction obtained from such construction. Mr. Kinney.

* Chicago, Ill.

## (11) WOOD BLOCK PAVEMENTS.

By Messrs. Frank W. Cherrington, Clifford Richardson, Edward Whitwell, W. H. Fulweiler, E. W. Stern, J. W. Howard, S. R. Church, Walter Buehler, H. W. Durham, and Ellis R. Dutton.

Mr. Cherrington.

Frank W. Cherrington,* Assoc. M. Am. Soc. C. E. (by letter).†—It is important that all the voids in a creosoted wood block pavement be thoroughly filled, in order to prevent the retention of organic substances and moisture deposited on the surface of the street. The report states that the blocks "should be laid closely and the joints filled with some suitable material." It is a well-established fact that, when wood blocks are laid closely together, it is practically impossible to fill the joints thoroughly with any material.

If the blocks are laid tightly together, very little filler will penetrate the interstices between them more than ½ in. below the surface.

In the writer's opinion, it is better to require that, as far as practicable, the spacing around each individual block shall average about ⅛ in., in order to enable any applied filler to penetrate the full depth of the joints.

Although two kinds of filler are favorably mentioned in the report, without a doubt, the most efficient, under all conditions, is a bituminous material. The only objections to it are the additional first cost of construction and the possibility that it may become objectionable in hot weather. The former may be answered by the increase of efficiency and the latter by the adoption of a standard bituminous filler, similar to that recommended by the American Society for Municipal Improvements. Care should also be taken to prevent the interstices between the blocks from being entirely filled. The best results are secured by filling the joints to within about 1 in. of the top, and using sand for the remaining space.

A ⅛-in. open joint requirement for a creosoted wood block pavement has several distinct advantages:

First.—It permits the bituminous filler to penetrate the full depth of the block on all sides, and thus provides a more compact and thoroughly water-proofed pavement.

Second.—A pliable expansion joint is provided for each individual block, thus allowing an additional factor of safety against possible destructive expansion.

Third.—It presents a less slippery surface to traffic by slightly separating the rows or courses of blocks.

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* Toledo, Ohio.

† Received by the Secretary, January 19th, 1917.

For these reasons the writer believes that a ⅛-in. open joint specification will give more satisfactory results than the provision in the report which states that the blocks should be laid closely. Mr. Cherrington.

Clifford Richardson,* M. Am. Soc. C. E. (by letter).†—As a preservative for wood block, the Committee concludes that "taking all things into consideration, coal-tar creosote oil is the best." Mr. Richardson.

This shows ignorance of the fact that the large areas of wood blocks, which have been treated with water-gas tar during the last decade, have proved that they are not only equal to similar blocks treated with coal-tar creosote, but, in many cases, superior. This has been demonstrated in the case of the experimental pavement laid by the Bureau of Highways, of the Borough of Manhattan, on Second Avenue. The material possesses the particular advantage that it does not bleed in the early years of its use under the hot summer sun.

The Committee calls attention to the European practice of laying the blocks directly on the concrete bed, but it fails to mention the method of grouting them, which is an essential feature. After the blocks are in place, the interstices are filled with coal-tar to a depth corresponding to half their thickness. This prevents water from acting on their lower surfaces. To prevent the effusion of the tar on the surface of the pavement, the upper half of the interstices is filled with a Portland cement grout. At various times, in technical journals, the writer has called the attention of American engineers to this practice, but, although it is such a desirable method of construction, it has not been adopted in America.

The Committee is to be congratulated on having assembled such a large mass of information in regard to road construction, and its report, when revised, will be of great use to highway engineers, although it cannot be looked on as possessing any finality, but must necessarily be amended from year to year in the light of experience.

Edward Whitwell,‡ Esq. (by letter).†—In Europe, particularly in England, the soft wood blocks are finding preference over hard blocks, because they are found to wear equally well, are just as cleanly, less slippery, and decidedly more economical. In their use the old sand cushion has been discarded, as it was found to be more desirable to secure an accurately laid bed than to rely on the cushion making up for the unevenness of the foundation; and there, as distinct from the Committee's views, an expansion groove at each side of the highway is considered absolutely essential. Mr. Whitwell.

The writer expected to find in the report some statement with regard to the relative values of the different forms of pavement, and

* New York City.

† Received by the Secretary, January 19th, 1917.

‡ London, England.

Mr. Whitwell. is of the opinion that a table showing the comparative values of the several forms, under identical conditions, would have been a great asset; but it may be that tests on national lines have not yet been carried out in America. If such national tests have not yet been carried out, it is evident that they should be, for they would result, as in other countries, in an important saving of time, money, and materials.

The report is excellent, and must commend itself to laymen and professionals alike, as it marks another very important step in the progress being made in the science of road construction.

Mr. Fulweiler. W. H. FULWEILER,* ASSOC. M. AM. SOC. C. E.—One point in connection with the Committee's report that should receive considerable attention before final adoption has to do with the proposed specifications recommended for "preserving oils" for use in the treatment of wood blocks.

The speaker does not wish to go into a discussion of the merits of the specification here proposed, but merely to question the advisability of adopting and publishing these specifications in the report.

Reports of Special Committees of this Society should be final and authoritative, and should represent, at the time of their adoption, the result of extensive investigations; they should be the best thought of the Profession; and, as such, they would not be a subject for immediate revision.

This Committee has been in existence for several years, and has, from time to time, published progress reports, showing the result of its labors, in which reports it has considered various types of pavement which have involved the use of bituminous materials. In none of these reports has an attempt been made to go into the subject of detailed specifications; but, on the contrary, they have set forth the more or less basic principles which should be observed in the selection of materials for different purposes. In the present state of the art, the speaker believes that this is the only course open to the Committee.

Now, however, this report presents these specifications for preserving oils for use in the treatment of wood blocks. It is difficult to understand why the Committee should single out specifications for these materials. Certainly, it cannot be on account of the greater importance of the materials, because, as compared with the construction of other forms of pavement, the yardage of wood block is but a small percentage.

Nor are these specifications due to any research work, or investigation on the part of this Committee, so that they do not involve any new principles; on the contrary, they are those that have been presented to several Societies, which have made it their business to study

* Wallingford, Pa.

the methods of testing, without which a specification is quite useless, and to develop and publish specifications for various classes of materials used in engineering. These Societies have standing Committees on specifications, whose duty it is to attempt to reconcile the requirements of the material and the commercial considerations underlying its supply, and to present to the Society from time to time such modifications of the existing specifications, or entirely new specifications, as the development of the art and the commercial availability of the materials warrant. Mr. Fulweiler.

This necessarily involves changes from time to time, and, as this continual change may be expected, due to conditions over which the Engineering Profession has no direct control, it would seem far better for this Committee to refer to the work being done by other Societies, rather than to publish a specification of such a type as included in this report.

To discuss more particularly the general form of these specifications, attention is called to the fact that they are not really specifications for preservative oils, but are descriptions. That is, the qualities required in a preservative oil for the treatment of wood blocks are not directly determined by the requirements of this specification. On the other hand, they merely serve to describe in technical language certain commercial articles of manufacture. They in no way resemble the specifications for iron, steel, cement, stone block, brick, etc., in which the specifications, by their requirements, secure directly, in the finished product, the desirable qualities that the product should exhibit in use.

Again, the particular specifications here proposed are so new that they were not even completed in the report sent out for discussion, and have never been subject to the test of practical usage.

In view of all of the foregoing, the speaker believes it is a serious mistake on the part of this Committee to include these specifications in its report, and it would be a still more serious mistake on the part of this Society to accept, and, thereby adopt, a specification for materials of this class, on account of the fact that the art has not arrived at such a state of development that a specification will secure the qualities that are essential in the material, as there is no general agreement as to just what these qualities are, or their limits; and, especially, the Society should avoid adopting an untried specification which, in the course of a year's practical use, may prove to be quite unsatisfactory.

It would seem to be far better for the Committee to confine its work to the presentation of "the fundamental principles on which specifications covering each of the several types of roads and pavement should be based." If it could, in this manner, secure data which

Mr. Fulweiler. would bring about a more general agreement as to the qualities that are necessary and the limits that are required, some permanent progress will have been made in the art.

Mr. Stern. E. W. STERN,* M. AM. SOC. C. E.—The wood block pavement has exhibited two defects, and these must be corrected in order to allow it to come into that use to which its valuable qualities should entitle it. These defects, as is commonly known, are bulging and bleeding. In the Borough of Manhattan there are examples of pavements laid according to suggestions made by the Committee; that is, there are some in which there is practically no expansion joint at all. The blocks are laid together as close as possible, a little sand being used to fill in the joints, and there are no expansion joints. Illustrating this type there may be cited, a pavement laid in 1911, on Lafayette Street, subjected to very heavy traffic. Owing to special local causes, practically no cuts have been made in this pavement; it has been left intact. The blocks had about 20 lb. of heavy oil. Not a cent has been expended for repairs on that street since it was laid, whereas there have been considerable repairs on wood block pavements on other streets laid at about the same time. The reason for this splendid record is due entirely to the fact that practically no cuts were made in the pavement (there is a school house on one side of the street and old "taxpayer" buildings on the other). The pavement is really water-proof. No water can get through the surface. To-day one has to scrape with a knife in order to find a joint, it has been compacted so well by the heavy traffic; from a little distance it looks like an asphalt pavement. Such conditions cannot be obtained, except in very rare cases, in the Borough of Manhattan. Many of the wood block pavements have bulged, and the speaker is of the opinion that therefore there should be a ⅛-in. joint around every block, and an expansion joint at each curb. A joint ⅛-in. wide is about the minimum, in order to permit the use of a bituminous filler, and the speaker believes in such a filler, and in a thorough filling of the joints.

Where old pavements with tight joints have been repaired, with the ⅛-in. joint, very good results have been secured, and there has been practically no bulging. There is a pavement on a certain street in which there have been eight bulges in the last few weeks, at the street intersections, some of them as high as 10 in.

The kind of joint to be used must depend on where the pavement is to be laid. In a city there should always be expansion joints, because there will always be cuts.

As to bleeding, on the theory that the blocks must be absolutely water-proof, it is necessary to put in a large quantity of preservative—about 20 lb.—and then bleeding is inevitable. Practical experience,

* New York City.

however, has shown that the pavement cannot be maintained as a water-proof structure on account of the settlement in the base due to cuts. It should be expected that the blocks will swell, and, therefore, expansion joints are necessary, both around the blocks and along the curbs. Mr. Stern.

A pavement was laid on 57th Street, New York City, during 1916 with a 16-lb. treatment of heavy oil. It had final steaming and vacuum treatment. This bled a little. It will not be known until next year whether it is satisfactory.

Bleeding is a very objectionable feature. There are so many complaints about this nuisance that it must be eliminated.

J. W. Howard,* Esq.—With reference to preservative oils for wood paving blocks, it should be possible to combine tar oils in one specification with one set of tests, so as to provide for water-proofing and germ-proofing qualities. Mr. Howard.

As some engineers believe that water-gas-tar-oil produces good results, it might be included with the other two, the three being combined in one general blanket specification, thus avoiding the objectionable alternate or special specifications for several brands and types.

The new specifications mentioned in the report were presented at the convention of the American Society for Municipal Improvements, in October, 1916. They represent a contention between commercial interests of oil producers. The purpose of this Society should be to avoid commercial influences and decide the qualities required and the methods of use for properly preserving wood. It is suggested that the Committee make a strenuous effort, with professional assistance of proper quality and character, to blend into one these specifications for two or even three oils.

Float tests have never been used in connection with preservatives of wood blocks by any city in the world; but only possibly in the laboratories of companies which prefer to make, if not practically control, one oil. The proposed "coal-tar-paving-oil" is not a distillate creosote oil, nor a true creosote oil.

The speaker suggests striking out all float tests in both oils. If such a test is adopted, however, there is no reason for specifying 10% and 0° cent. for the "distillate oil", and 35% and 100° cent. for the "coal-tar-paving-oil." They should be for the same percentage and at the same temperature, to be determined by proper non-commercial chemists, thus having only one variable, and that should be the time, in seconds, for the float test (the time for the float to sink). The speaker suggests inserting the long-used and good test that "the residue remaining after distillation up to 355° cent. should be soft and easily.

* New York City.

Mr. Howard. indented with the finger at 77° Fahr.", or that a definite needle penetration minimum test be established and inserted.

All tests which are descriptions of a particular brand or kind, should be omitted, and only the needed qualities for preserving wood blocks should be in the specifications, and such as to permit the use of all good creosote oils of the general type required.

The speaker believes that the proposed new specifications have never been used, and that they are descriptions (in new language and with new tests) of the same heavy tar-containing oil as those heretofore described in specifications by other tests for the "coal-tar-paving-oil"; and, as to the "distillate oil", that the words and tests are largely new and untried, but cover one (but not all) good distillate creosote oil heretofore used under other and standard tests.

These new tests preclude the use of valuable city and other records of tests made by the old standard methods of successful oils, accumulated in many cities and countries; and they prevent the comparison of newly offered oils, tested by new tests, with those records.

As to the inspection of the manufacture of the blocks at the plant, the provision is good; but because small towns and other purchasers of small quantities cannot send inspectors to plants, the words, "if possible", should be inserted after the words, "be subject", in the second line of the next to the last paragraph on "Inspection", on page 1640. Furthermore, at the top of page 1641, the words "except that the plant inspections should be final with respect to the kind of wood, rings per inch, oil, and treatment" should be omitted. No city or chief engineer should or would give up his right, and sometimes the necessity, of inspecting the paving blocks himself after they had arrived in a city or the place to be used, or of making a re-inspection to check or control a subordinate stationed at a distant plant where the blocks had been made.

Mr. Church. S. R. CHURCH,* ESQ.—In 1913 the speaker devoted considerable attention to the inspection of wood pavements in European cities. The standard method of construction there was to set the wood blocks on a smooth concrete foundation, then flush pitch over the surface, and squeegee it into the joints at the top, completely filling them.

The speaker did not see any top joint filler of cement-grout used, either in London or other cities, nor is it part of the specifications of the Borough of Westminster or the old City of London.

Mr. Howard, in discussing the methods of analysis in connection with the preservative specification, attempts to discredit the float test, and states that it is better to determine whether or not the residue is soft by specifying that it shall be soft, and testing it by pressure with the finger. He says that the float test is inaccurate.

* Mgr., Research Dept., The Barrett Co., New York City.

The speaker submits that the float test is more accurate than any test that can be made with the finger, and thinks that the Committee made a step in advance in specifying a definite means of determining consistency, in place of the old requirement that "the residue shall be soft." Mr. Howard suggests that for accuracy a needle penetration test be made. However, the peculiar adaptability of the float test to these residues was determined after comparative tests in a number of laboratories, these tests including needle penetration, float, melting point, and two or three other methods. Mr. Church.

WALTER BUEHLER,* M. AM. SOC. C. E.—Wood block pavements are subject to two defects, namely, expansion and bleeding. If the Committee's recommendations are followed, both these defects will be reduced to a minimum. For a number of years the character of the oil used in preserving the blocks has been held responsible for the bleeding, but the speaker is convinced that it has absolutely nothing to do with it. The method of treatment, character, and condition of the wood at the time of treatment are of greater importance. Mr. Buehler.

On page 1639 of the report, under the heading "Treatment of Wood Blocks," it is stated that either air-seasoned or green timber may be used, but that it should preferably be treated within 3 months from the time it is sawed. "Green timber and seasoned timber, however, should not be treated together in the same charge." From the speaker's knowledge, that has not been the practice in all timber-preserving plants. Too little attention has been given to the separation of the green and seasoned timber. Then the report states: "In all cases, whether thoroughly air-seasoned or green, they should first be subjected to live steam." In one case, which came to the speaker's attention in the summer of 1916, the specifications contained this clause, and yet the plant operator put the oil into the blocks without the preliminary steaming. His reason for it was that the wood was seasoned so thoroughly and the oil went in so easily that there was no necessity for this preliminary steaming. Trouble is invited from expansion whenever that is done. Bearing in mind that Southern pine can be treated with as much as 28 lb. of oil per cu. ft., one can realize the importance of a specification containing the time element as pertaining to the pressure used in forcing in the oil, in other words, the importance of securing a uniform distribution of oil throughout the block. If a pressure of 200 lb. is applied immediately after the cylinder is filled, it is possible to put in 16 lb. of oil per cubic foot, in a comparatively short time, but that oil will be near the surface of the blocks and, when subjected to the summer heat, is bound to expand and come out. If 16 lb. of oil is put in under a pressure of 50 lb. during a period of, say, 5 or 6 hours, or if it is put in in accordance with the

* Chicago, Ill.

Mr. Buehler. recommended specification, it will be distributed uniformly throughout the block, and there will not be a volume of oil near the surface.

On page 1640, the last step in the process is given, as follows:

"After this a supplemental vacuum, in which the maximum intensity reached is at least 20 in. and the time the vacuum is applied not less than 30 min., should be applied. If desired, this vacuum may be either preceded or followed by a short steaming period."

This, apparently, is an infringement on a patent recently granted, but as the patent is dependent on a definitely stated sequence of operations, a slight change in the wording will overcome any danger of infringement. The patent provides for five distinct steps, in the following sequence:

First, steaming;
Second, vacuum;
Third, oil pressure;
Fourth, heating by steam, or otherwise;
Fifth, vacuum.

By changing the sequence of the last two steps, the results obtained are just as effective, and there can be no infringement. In other words, all that is necessary is to eliminate the words "either preceded or." As amended, the last sentence of the paragraph referred to would then read: "If desired, this vacuum may be followed by a short steaming period."

In reference to joint fillers, the speaker agrees that blocks should not be laid tight, but he believes that it is not practical to secure a space of ⅛ in. around each block. There are blocks on the market with lugs to provide this spacing, but, as these are patented, the speaker does not desire to discuss their merits at this time.

If blocks, as generally manufactured, are in proper condition for laying, are laid tightly by hand, and a pitch filler, heated to not less than 250° Fahr., is used, there will be no difficulty in sealing each joint thoroughly. The method used in applying the filler is of extreme importance. In the first place, it must be heated to a temperature of not less than 250° Fahr. and not more than 300° Fahr., and poured over the surface of the pavement, and squeegeed into the joints. The hot filler is expanded and on cooling will contract, leaving the top of the joint open to be filled with sand (preferably hot). The mistake is sometimes made of attempting, by a second application, to fill the empty space left by the contraction of the filler when first applied, with the result that most of the pitch used in the second application remains on top of the pavement. It is important not to fill the upper parts of the joints, but to have their lower parts filled, so that each will be sealed thoroughly, and thus prevent moisture from getting under the blocks.

H. W. DURHAM,* M. AM. SOC. C. E.—When the speaker was in Europe in 1913, he investigated some of the wood block pavements. On the Strand, in London, a coat of pitch was applied to the surface of the concrete foundation, and the blocks were laid on that. Filler was then squeegeed over them into the joints, and after that had had a little time to harden, a thin liquid cement grout was swept over the entire surface and covered with fine crushed screenings. The apparent purpose of the grout surface was merely to take away the stickiness of the coal-tar. That was also seen in French practice, and in some of the cities of Holland. It seemed to be a common method of laying a soft wood pavement. Mr. Durham.

Attention should be called to the fact that wood pavements abroad are of a softer pine than in America. The wood is of a different kind. If European methods are exclusively advisable, it will be necessary to import the kind of wood used there.

The test pavements on Second Avenue, New York City, were laid in 1912 under the speaker's direction, and since they were laid, he has inspected them rather closely. He has been unable to determine any very great or appreciable difference in the quantity of bleeding with different types of treatment or materials with which the blocks were treated. During the first few months, the bleeding from some blocks was greater than from others, but it is not fair to say that the water-gas tar blocks bleed more or less than the others. In general, it is very difficult to discover any difference, unless one has a diagram showing where the different sections are laid.

Attention has been drawn to a short, but very good, section of wood pavement on Lafayette Avenue, and the statement has been made that no cuts were ever made there. There have not been many, but a very large one was made a few years ago nearly from curb to curb transversely to the axis of the street. It is a tribute to the inspector who had charge of that work, that the present Chief Engineer states that he is unable to find evidence of any cuts. It indicates that it is possible to restore a cut, if enough attention is paid to it, so that the pavement will be substantially as before.

Wood pavements have been laid in the Borough of Manhattan under a great many different specifications, as has been stated, but it is hardly fair to claim that tight joints and any particular class of construction are responsible for the blocks swelling or traveling. On Broadway north from Columbus Circle, the wood pavement is laid on a sand cushion, with sand in the joints, and the blocks tight, as has been mentioned, and this is a very good pavement, in good condition to-day, and apparently it will be good for a long time to come.

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* New York City.

Mr. Durham. On the other hand, without in any way intending to detract from Mr. Stern's improvements or make any particular criticism of his specifications, the speaker noticed last summer that in the new wood block pavement on 57th Street, in front of the Society House, which has been laid with ⅛-in. joints, there was considerable bleeding. If there are such joints in a pavement, they will probably allow so much play that the blocks will shove or travel.

Mr. Dutton. ELLIS R. DUTTON,* ESQ. (by letter).†—On page 1639, the words "at the discretion of the treating plant operator" should be eliminated, because the purchaser of the blocks would not have anything to say in regard to the time for steaming them prior to the injection of the oil; and the purchaser is the one who is paying for the blocks, and has to suffer if they are not properly treated. It is not right for the seller to dictate what the treatment should be under a specification.

On page 1641 the statement that "the plant inspection should be final with respect to the kind of wood, rings per inch, oil, and treatment," entirely precludes the right of the purchaser to further inspection of the blocks. It would be almost impossible for the purchaser to provide enough inspectors to look after all these things. The inspection at the plant should cover only the oil, the quantity of oil per cubic foot injected, and the method of treatment. These items could not be inspected by the purchaser after the blocks were received on the street; but the kind of wood and the rings per inch could be inspected as well there as at the plant. Therefore, if the final inspection included these two items, the contractor would be more careful in providing the proper kind of wood and the rings per inch, at the plant. Otherwise, the report is very satisfactory, and the writer is glad that this Society and the other societies dealing in such matters have met on a common ground, and are able to decide on practically standard specifications.

* Asst. City Engr., Minneapolis, Minn.

† Received by the Secretary, January 19th, 1917.

# MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

## WILLIAM DOUGLAS PICKETT, Hon. M. Am. Soc. C. E.*

DIED MARCH 5TH, 1917.

William Douglas Pickett was the son of George Blackwell Pickett and Courtney (Heron) Pickett, members of prominent Virginia families. After their marriage, the Picketts had settled in the Tennessee River Valley, near Huntsville, Ala., where William Douglas Pickett was born on October 2d, 1827. After the death of his father, the family moved to Kentucky where the boy received his preliminary education, first in Richmond and then in Lexington. Having chosen Civil Engineering as his life work, he prepared himself for it at Transylvania University, from which he was graduated in 1845.

Immediately after his graduation, Mr. Pickett went to Texas, where he was engaged as Engineer in establishing the disputed lines of a large landed estate. While thus employed, war with Mexico was declared, and Mr. Pickett enlisted with the Texas Rangers. In 1848 when the war was ended, he returned to Kentucky and resumed the practice of his Profession. He was engaged on the reconstruction of the strap-railroad from Lexington to Frankfort, the first railroad built west of the Alleghany Mountains. He also served as Assistant Engineer on the railroad constructed from Lexington to Danville, and, later, as Chief Engineer, he located and built railroads in Tennessee, Georgia, Alabama, and Arkansas.

At the outbreak of the Civil War in 1861, Mr. Pickett was employed as Chief Engineer on the construction of the Memphis and Ohio Railroad in Tennessee. He resigned immediately and enlisted in the Confederate Army. He raised a company of civil engineers and received a commission as its Captain from the Governor of Tennessee. Shortly afterward, he was transferred to the staff of Major-General Pillow and designed and constructed a number of water batteries along the Mississippi River. While serving as Engineer Aide to General Leonidas Polk, Captain Pickett was requested by General Polk to accompany him to Columbus, Ky., to witness the test of a new gun. On the first discharge, the gun burst, killing a number of the men who served it, the concussion hurling both General Polk and Captain Pickett through the air. Although he escaped with his life, Captain Pickett's hearing was permanently impaired by the shock. He was afterward transferred to the staff of Major General Hardee as Military Engineer, and took

* Memoir prepared by the Secretary, from information on file at the House of the Society.

part in the battles of Shiloh, Perryville, Murfreesboro, and Missionary Ridge, as well as the engagements around Atlanta.

When General Hardee was transferred to a new Department, with headquarters at Charleston, S. C., Captain Pickett went with him, and was actively engaged in the siege of Savannah, the evacuation of Charleston and Fort Sumter, and the march of Hardee's Army to a junction with that of General Johnston, and, on the surrender of the latter, in April, 1865, Captain Pickett was paroled as Colonel and Inspector General of Hardee's Division.

After a venture in cotton planting, unsuccessful on account of floods in Yazoo Pass, Colonel Pickett resumed the practice of his profession, and, as Chief Engineer, rebuilt the bridges and reconstructed the roadbed for the Memphis and Ohio Railroad Company, now a part of the Louisville and Nashville System.

On April 27th, 1870, Colonel Pickett was married to Miss Theodosia Curd, of Lexington, Ky., who died a year later, leaving a son who survived her only four years.

After the deaths of his wife and little son, Colonel Pickett went West in 1873, and after several years spent in prospecting and fighting Indians over the territory now embraced in the Yellowstone Reservation, he established himself on a cattle ranch in the Big Horn Country, in Wyoming, and became widely known as a successful hunter of the grizzly bear. When this region became populated and prosperous, Colonel Pickett represented his district for several terms in the State Legislature of Wyoming, first as Representative and afterward as Senator.

In 1904, he returned to Kentucky and purchased a home in Lexington, where he lived until his death on March 5th, 1917, in his ninetieth year. Knowing the hopelessness of his case, he met his end like a soldier, calmly and unflinchingly, his last days being disturbed only by the thought of leaving his invalid brother, Major George Blackwell Pickett, who survives him.

A life-long friend, Judge George B. Kinkead, of Lexington, Ky., writes of Colonel Pickett as follows:

"Few knew him intimately, but to many the tall, alert figure of this handsome old gentleman, democratic in his feelings, but displaying the patrician in every lineament, was familiar. In every walk of life, his most striking and distinguishing characteristic was a prompt recognition of every duty imposed, and an unfaltering courage to discharge it. One who lived close to him for years, but a few days since remarked to the writer, 'He was the best neighbor I ever knew.'"

Colonel Pickett was a member of the Boone and Crockett Club, and the Confederate Veterans' Association. He was also a member of Christ Church Cathedral, from which his funeral was held on March 7th, 1917.

He was the oldest living member of the American Society of Civil Engineers and although not present at the meeting of November 5th, 1852, when the Society was organized, he was elected a Member on July 6th, 1853, and therefore might be classed as a Charter Member. Throughout his long membership, he took a lively interest in the welfare and prosperity of the Society and on all occasions proudly displayed his badge. In 1908, Colonel Pickett, then in his eighty-first year, contributed a discussion on the paper by H. M. Chittenden, M. Am. Soc. C. E., entitled "Forests and Reservoirs in Their Relation to Stream Flow with Particular Reference to Navigable Rivers"*, and also a paper entitled "The Floods of the Mississippi Delta: Their Causes, and Suggestions as to Their Control".† In 1911, Colonel Pickett attended the Annual Convention of the Society, which was held that year at Chattanooga, Tenn.

Colonel Pickett was elected an Honorary Member of the American Society of Civil Engineers on April 1st, 1914.

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## JOHN FERRIS ALDEN, M. Am. Soc. C. E.‡

Died February 27th, 1917.

John Ferris Alden, the son of Sidney and Harriet Webster Alden, and a direct descendant in the eighth generation of John Alden, of Plymouth, Mass., was born at Cohoes, N. Y., on March 19th, 1852. He was educated in private schools in Albany, N. Y., and at Rensselaer Polytechnic Institute, from which he was graduated with honors, in June, 1872, with the degree of Civil Engineer.

Mr. Alden did his first engineering work as Assistant Engineer on the construction of the railroad bridge to carry the tracks of the New York Central Railroad over the Hudson River at Albany, N. Y., under Mr. Charles Hilton as Chief Engineer.

In January, 1875, Mr. Alden accepted a position as Assistant Engineer at the Leighton Bridge and Iron Works at Rochester, N. Y. In 1877, he was made Chief Engineer and, in 1878, became a member of the firm. On July 1st, 1881, with the late Moritz Lassig, M. Am. Soc. C. E., as partner, he leased the Leighton Bridge and Iron Works and continued the business in Rochester under the name of Alden and Lassig and that in Chicago, Ill., under the name of Lassig and Alden.

In January, 1886, the partnership was dissolved and Mr. Alden purchased the business interests of the firm in Rochester, Mr. Lassig

* *Transactions*, Am. Soc. C. E., Vol. LXII, p. 423.

† *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 53.

‡ Memoir prepared by the Secretary, from information on file at the House of the Society.

retaining those in Chicago. Mr. Alden then re-organized his business under the name of the Rochester Bridge and Iron Works of which he continued as sole owner until 1901 when he sold out to the American Bridge Company. He retained an interest in the Company, however, until his death.

During his active connection with the Rochester Bridge and Iron Works, Mr. Alden built many steel and iron bridges, especially for railroads, throughout the United States and Canada. Among these may be mentioned the Elevated Railroad in New York City; bridges for the Delaware and Hudson Railroad, the Chicago, Milwaukee and St. Paul Railroad, and the Buffalo, Rochester and Pittsburgh Railroad; the bridge over the Columbia River at Pasco, Wash.; the viaducts at Los Angeles, Cal.; the Upper Suspension Bridge at Niagara Falls; and the Driving Park Avenue Bridge at Rochester, N. Y. He also constructed the tower and elevator in the House of Parliament, at Ottawa, Ont., Canada, as well as many buildings in New York, Chicago, and other cities, and furnished most of the iron and steel work for the buildings of the Columbian Exposition, at Chicago, Ill.

Mr. Alden was married, in 1885, to Miss Mary E. Bogue, of Brooklyn, N. Y., who, with two sons and three daughters, survives him.

At the time of his death which occurred at his home at Rochester, N. Y., on February 27th, 1917, after a brief illness, Mr. Alden was President of the Locke Insulator Manufacturing Company and a Director of the Genesee Valley Trust Company. He was also a member of the Rensselaer Society of Civil Engineers, the Rochester Chamber of Commerce, the Alden Kindred of America, the Sons of the Mayflower, the Genesee Valley Club, and the Rochester Club. He was Vice-President of the Board of Trustees of the Friendly Home, and was actively interested in the erection of its new home. He was also a Vestryman and prominent member of Christ (Protestant Episcopal) Church.

Mr. Alden was elected a Member of the American Society of Civil Engineers on July 6th, 1887.

## RICHARD EVANS, M. Am. Soc. C. E.*

Died December 30th, 1916.

Richard Evans was born at Caracas, Venezuela, on November 15th, 1855, and, on his father's side, came from the Evans family long established in New England. While he was quite young, his parents removed to the United States and settled in Woodbury, N. J. Here he received his preliminary education before entering the Polytechnic

* Memoir prepared by the Secretary from information on file at the House of the Society.

College of Pennsylvania, from which he was graduated with the Class of 1875.

In 1876, Mr. Evans was employed as Rodman and Levelman on the location of the Philadelphia and Atlantic City Railroad, and, from 1877 to 1879, he was engaged as Levelman and Transitman on topography and revision of lines and grades in the Twenty-third Ward for the City of Philadelphia. In 1879, he was appointed Transitman, and, later, Assistant Engineer on the preliminary and location surveys for the Danville and Shamokin Railroad, in Pennsylvania, and, in 1880, he was made Assistant Engineer on the location of the Five Mile Beach Railroad and laying out the Town of Anglesea, N. J. From July, 1880, to February, 1881, he served as Transitman in the Maintenance of Way Department of the Pennsylvania Railroad.

In 1881, Mr. Evans went to Mexico as Transitman and Chief of Field Party on the location surveys for the Mexican National Railroad, returning in 1882, to become Assistant Engineer on the New River Branch of the Norfolk and Western Railroad. In 1883, he was appointed Assistant Engineer with the Pennsylvania Schuylkill Valley Railroad Company, on the revision of part of its line. He designed and made the plans for arch culverts, piers, and bridge abutments on 6 miles of heavy construction, including the Schuylkill River crossing and viaduct approach.

From 1884 to 1887, Mr. Evans was engaged with the Survey Department of the City of Philadelphia on sewer construction in the Manayunk District and on laying out new streets in the Twenty-second Ward.

In 1887, Mr. Evans went West and was employed as Assistant Engineer with the St. Louis and San Francisco Railroad Company. Returning to the East in 1888, he was engaged until 1890 in private practice, making plans, estimates, and superintending the construction of macadamized and telford roads, highway bridges, etc., in Montgomery County, Pennsylvania. Afterward, he accepted for a short time a position as Principal Assistant Engineer on the Philadelphia and Seashore Railroad.

In July, 1890, Mr. Evans established himself in private practice at Hagerstown, Md., in partnership with Mr. C. C. Vandevanter, which was continued until 1893. He then removed to Jamaica, N. Y., where he formed a partnership with his brother, Mr. C. A. Evans, and continued in private practice as a Consulting Engineer until his death, which occurred on December 30th, 1916. He was married in Philadelphia, Pa., about 1887, but his wife died two years later.

In his work and conduct, which was always above criticism, Mr. Evans exemplified the best influences of his profession. Realizing his incompatibility with the prevailing requirements of corporation service as he saw them, he preferred the private practice of engineering for

which he felt himself better fitted temperamentally. Holding to that which he considered to be right, his decisions in regard to his work were made regardless of who was or was not benefited thereby. In the study of any contemplated project, he always showed that calmness, ease, and continuity of application which distinguish the trained engineer.

Mr. Evans was elected a Member of the American Society of Civil Engineers on June 7th, 1893.

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## WILLIAM OSWALD HENDERER, M. Am. Soc. C. E.*

Died February 8th, 1917.

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William Oswald Henderer, the son of Myers Henderer and Euretta Curtis Henderer, was born on May 11th, 1865, in Greenbush, now known as Rensselaer, Rensselaer County, N. Y. He received his early education in the district schools of Schodack Landing and Castleton, in Rensselaer County, and in the grammar and high schools of Albany, N. Y. His technical training was received at Rensselaer Polytechnic Institute, Troy, N. Y., where he was a member of the Class of 1887.

From June, 1887, to January, 1888, Mr. Henderer served as an Assistant in the office of the City Engineer, of Troy. During the spring and summer of 1888 he worked on a preliminary survey through Northern Ohio for the Pennsylvania Railroad Company.

From May, 1888, until April, 1891, he was with G. W. G. Ferris and Company, of Pittsburgh, Pa., as Shop Inspector on bridge, railroad, and viaduct steelwork, and it was here that he became acquainted with Frank C. Osborn, M. Am. Soc. C. E., with whom, later, he was associated.

From April to November, 1891, Mr. Henderer was Draftsman and Assistant Engineer for the Detroit Bridge and Iron Works, and from November, 1891, to March, 1893, he served in a similar capacity with the Berlin Iron Bridge Company, of East Berlin, Conn.

In March, 1893, he went to Cleveland, Ohio, to be with Mr. Osborn, in charge of inspection and tests. From December, 1897, to June, 1900, as a member of the Osborn Company, Civil Engineers, he was in charge of structural design, inspection, and general office work. During this period, the Brooklyn-Brighton Viaduct and the South Rocky River Viaduct, in Cuyahoga County, Ohio, and the Zanesville Y-Bridge, in Zanesville, Ohio, were among the larger and more interesting works handled by the Company.

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* Memoir prepared by Kenneth H. Osborn, Assoc. M. Am. Soc. C. E.

In June, 1900, The Osborn Engineering Company was incorporated, and Mr. Henderer became its first Vice-President. This position he held until the summer of 1910, at which time Mr. Osborn retired from active charge of the company, and Mr. Henderer became President.

In recent years, the Boston, New York, Cleveland, Washington, and Detroit baseball parks, and the factory buildings for the Firestone Tire and Rubber Company and The B. F. Goodrich Company, have been included among the more important structures designed by the Company.

Mr. Henderer continued as President of the Company until his death, on February 8th, 1917, at Miami, Fla. He had been absent from Cleveland only about 10 days on a fishing trip, when he was seized by a sudden attack of peritonitis. He was taken to a hospital in Miami, but lived only a few days after the attack.

Mr. Henderer was married on June 21st, 1893, to Ida F. Mayer, of Cleveland, and had one daughter, Geraldine.

He was an active member of the Cleveland Engineering Society, and had served as President of that organization. He was also a member of the American Society for the Promotion of Engineering Education, the Cleveland Athletic Club, the Shaker Heights Country Club, and the Chippewa Lake Club. He was deeply interested in the Masonic organizations of Cleveland, and was very active in all those of which he was a member.

Mr. Henderer was elected a Member of the American Society of Civil Engineers on April 3d, 1901.

## JOSEPH RAMSEY, Jr., M. Am. Soc. C. E.*

Died July 7th, 1916.

Joseph Ramsey, Jr., the son of Joseph and Mary (Patterson) Ramsey, was born at Pittsburgh, Pa., on April 17th, 1850. He was educated in the public schools of Pittsburgh, and completed his studies at the Western University of Pennsylvania in 1869.

Mr. Ramsey began his railroad career as a Rodman in the Engineer Corps of the Pittsburgh, Cincinnati and St. Louis Railroad. In 1870 he was made Engineer in charge of the construction of the Dresden Cut-off, at Dresden, Ohio, but resigned to accept the position of Assistant Engineer on location and construction of the Bell's Gap Railroad, and, a year later, was made Assistant Engineer of the Lewiston Division of the Pennsylvania Railroad.

* Memoir prepared by John P. Ramsey, Esq.

Returning to the Bell's Gap Railroad, in 1872, Mr. Ramsey was made Chief Engineer, and after its completion, in 1873, its Superintendent, in which position he continued until January, 1879, when he entered the service of the Pittsburgh, New Castle and Lake Erie Railroad as Chief Engineer and General Superintendent. He resigned this position in September of that year, to take a similar one with the Pittsburgh Southern Railroad. Mr. Ramsey's unusual abilities were speedily recognized, and he was offered the position of General Manager of the Pittsburgh, Chartiers and Youghiogheny Railroad. While thus engaged, he became General Manager of two other branch roads and the Chartiers Block Coal Company.

From 1883 to 1890 Mr. Ramsey served as Chief Engineer of the Cincinnati, Hamilton and Dayton Railroad, resigning to become Assistant to the President of the Cleveland, Cincinnati, Chicago and St. Louis Railroad, which post he retained until 1892. From 1890 to 1895 he was President of the Peoria and Pekin Union Railway and, from 1891 to 1893, Vice-President of the Cincinnati, Wabash and Michigan Railroad, having charge of the Operating and Traffic Departments of both these companies, while holding the Presidency of the Findlay Belt Railway. In June, 1891, he was made General Manager of the Cleveland, Cincinnati, Chicago and St. Louis Railway, and, in 1892, was elected General Manager and Vice-President of the Dayton Union Railroad. In April, 1893, Mr. Ramsey accepted the position of General Manager of the Terminal Railroad Association of St. Louis, which position he held until December 1st, 1895, when he was made Vice-President and General Manager of the Wabash Railroad Company. He was made President of the entire Wabash System in 1901, a position which he filled for five years with signal success.

Mr. Ramsey was also President of the Wheeling and Lake Erie Railroad and the Wabash-Pittsburgh Terminal Railroad from 1901 to 1905; the Ann Arbor Railroad from 1902 to 1906; and the Western Maryland Railroad and West Virginia Central Railroad from 1903 to 1905. He also served as President of the Pittsburgh and Chicago Railroad, the Lorain and Ashland, and the Ashland and Western Railways. In 1913, Mr. Ramsey became President and General Manager of the Lorain, Ashland and Southern Railroad, which he had built. While President of the Wabash Railroad he was sent to the Paris Exposition of 1900 as Railroad Commissioner from the United States.

Mr. Ramsey was a Director of the Louisiana Purchase Exposition, at St. Louis, Mo., and Vice-President of its Committees on Electricity and Electrical Appliances and Transportation. He was also connected with several companies as Director and Committeeman. He was a member of the Commercial and Noonday Clubs, of St. Louis, Mo., the Duquesne Club, of Pittsburgh, Pa., and the Toledo Club, of Toledo, Ohio.

Mr. Ramsey was married on April 8th, 1873, to Laura, daughter of James E. Palmer, of Zanesville, Ohio, who, with two sons, James Palmer and John Patterson, and three daughters, Mrs. Helen R. Fowler, Miss Jane M. Ramsey, and Mrs. Mary R. McIntyre, survives him. His death occurred at East Orange, N. J., on July 7th, 1916.

Mr. Ramsey was elected a Member of the American Society of Civil Engineers on May 1st, 1889, and served as a Director from 1900 to 1902, inclusive.

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## WILLIAM FREDERICK ALFRED ANSON, Assoc. M. Am. Soc. C. E.*

Died July 17th, 1916.

William Frederick Alfred Anson, the son of the Rev. Alfred W. Anson, was born at Staunton, Va., on August 7th, 1878. He received his early education under governesses and tutors in his home and in England. In 1893, he entered the Episcopal High School, at Alexandria, Va., where he remained until 1896. He then entered Roanoke College, at Salem, Va., where he remained for one year, after which he entered the Virginia Military Institute, at Lexington, Va. He attended the Institute for three years, but was unable to complete his course on account of ill-health. After leaving school, he remained on his father's farm until 1902.

In July, 1902 Mr. Anson entered the employ of the Norfolk and Western Railway Company, as Instrumentman and Inspector on construction work. He remained with this Company until 1907. He then went to Martinsville, Va., where he was engaged in private surveying until 1908.

He was then appointed Inspector in Campbell County under the Virginia State Highway Commission, and, later, was promoted to Resident Engineer by the Commission, with headquarters at Pulaski, Va., having charge of the work in several counties. In April, 1914, he was again promoted by the Commission to be County Engineer of Russell County, with headquarters at Lebanon, Va.

For several years before his death Mr. Anson had been in poor health, and in January, 1916, he went to Richmond where he was examined by a specialist who pronounced his trouble to be chronic appendicitis. He returned to his home in Lebanon, but soon decided to undergo an operation, and, in February, 1916, was operated on at the Bluefield Sanatorium, Bluefield, W. Va. He never fully recovered from this operation, although he returned to his work which he carried on until about a week before his death.

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* Memoir prepared by A. H. Pettigrew, Esq.

In his professional career Mr. Anson developed those qualities which insure success. He had been connected with the highway work in Russell County only a short time, but he was highly thought of by all the citizens, and the County road authorities were more than satisfied with his management.

In October, 1915, Mr. Anson was married to Miss Vera Seay, of Eagle Rock, Va. He is survived by his widow, his father, brother, and nine sisters.

Mr. Anson was regarded as an engineer of rare ability, and his work with the Virginia State Highway Commission has always been highly commended. He was a man of high principles and of strong convictions, never hesitating to stand up for what he thought to be right or to oppose that which he considered wrong. He was a devoted member of the Protestant Episcopal Church, which he had joined in his youth, and he will be greatly missed by his family and his friends and associates.

Mr. Anson was elected an Associate Member of the American Society of Civil Engineers on October 1st, 1912.

## GEORGE EDWARD VANSITTART, Assoc. M. Am. Soc. C. E.*

DIED MAY 14TH, 1916.

Major George Edward Vansittart, 13th Battery, 4th Canadian Field Artillery Brigade, died on May 14th, 1916, from shell wounds received while on active service somewhere in France.

The only son of John Pennyfather Vansittart, Public Works Department, India, George Edward Vansittart was born at Mussooree, India, on October 7th, 1884. He came of a long line of naval and military ancestors, on both sides. His great-grandfather and his uncle on his father's side were Admirals in the British Navy, and Bisham Abbey, in Berkshire, was for many years the home of his great-grandfather, of which branch of the family he was the last young male representative. His great-grandfather on his mother's side was Colonel Alexander Light who commanded with distinction in India, and elsewhere, the 25th Queen's Own Borderers.

George Edward Vansittart came to Canada at an early age, and was a graduate of the Royal Military College of Canada and also of McGill University. At the conclusion of his University career, in 1906, he adopted the profession of Civil Engineering. During the next two years he was employed as Resident Engineer on the construction of the Midland Railway of Manitoba and Assistant Engineer

* Memoir prepared by F. H. Peters, Assoc. M. Am. Soc. C. E.

on the Canadian Pacific Railway Irrigation Project, at Calgary, Alberta.

In the spring of 1909, Mr. Vansittart was engaged by Messrs. Smith, Kerry and Chace, Consulting Engineers, of Toronto, Ont., as Assistant Engineer on the construction work of the Crane Falls Power and Irrigation Company, of Boise, Idaho. Before severing his connection with this Company to enlist for active service, he held the position of Vice-President and General Manager.

When the Great War broke out, this gallant officer returned to Canada and enlisted as a subaltern in the Field Artillery, where his great ability gained for him rapid promotion, and, in a short time, he was gazetted as Major and given command of a battery. By the time his battery reached England, it was considered to be the smartest and most efficient unit in the artillery of the division to which it belonged. Decorations and further rapid promotions were prophesied for Major Vansittart, whose untiring energy, able judgment, and sound gunnery had impressed every one who came into contact with his work. A Canadian Staff Officer writing a personal letter said:

"Vansittart is the best battery commander of the outfit, and his gun positions are the pride of the district. Neighboring artillerymen are sent down to see them as a model of what gun positions should be. The enemy have never located them."

Major Vansittart was mentioned in a despatch from General Sir Douglas Haig, published June 15th, 1916, for gallant and distinguished service in the field.

George Edward Vansittart was elected an Associate Member of the American Society of Civil Engineers, on March 5th, 1912. He was also an Associate Member of the Canadian Society of Civil Engineers.

## GEORGE HENRY FROST, Assoc. Am. Soc. C. E.*

Died March 15th, 1917.

George Henry Frost, the son of Ebenezer and Caroline (Harwood) Frost, was born on July 9th, 1838, at West Hawkesbury, Ont., Canada, where his father was engaged in the foundry business. His parents were of New England ancestry, natives of Vermont, who had emigrated to Lawrence County, New York, and, about 1836, had removed to Canada. When Mr. Frost was about a year old, his parents removed to Smith's Falls, Ont., where the boy obtained his early education at the village school. In 1854, he was sent to an academy in Glover,

* Memoir compiled by the Secretary from information on file at the Society House.

Vt., and on his return to Smith's Falls, he taught school while preparing for college. He was graduated from McGill University in 1860 with the degree of Civil Engineer, and, at the time of his death, was its oldest living alumnus.

After his graduation from McGill, Mr. Frost, as was the custom in those days, served his apprenticeship to a licensed land surveyor, until 1863, when he received his diploma as a Provincial Land Surveyor. His ambitions were not satisfied, however, by the opportunities offered him at his home in Smith's Falls, and, in August, 1863, he went to Chicago, Ill., where he secured employment as Rodman on a railroad survey then being made by the Chicago and Northwestern Railroad in Wisconsin. He remained with that Company, first in the office of the Chief Engineer and afterward in the Land Commissioner's office, until 1868, except for the summer of 1864 when he was employed in an architect's office in St. Louis, Mo.

After leaving the employ of the Chicago and Northwestern Railroad Company, in 1868, Mr. Frost engaged in the private practice of engineering with an office in Chicago. After the great fire in October, 1871, he did a great deal of engineering work in connection with the rebuilding of the city and in the establishment of property lines within the burned district. Many of the suburban sections which have been incorporated as part of the City of Chicago were laid out by Mr. Frost at that time. His work included also the survey of the tracts which now comprise the United States Reservation at Fort Sheridan and the Town of Glencoe.

In April, 1874, Mr. Frost established and issued the first number of the *Engineer and Surveyor,* a monthly publication devoted to the interests of the civil engineer, and the first periodical of its kind in America. In 1876, the publication was changed to a weekly and re-named the *Engineering News.* In December, 1878, Mr. Frost moved his business to New York City and opened an office in the Tribune Building. Later, he transferred his office to the St. Paul Building. The *Engineering News* steadily prospered under his management until it became known as one of the leading authorities on civil engineering and allied subjects. In 1911, Mr. Frost sold his establishment to the Hill Publishing Company, of New York City, which recently was merged with the McGraw Publishing Company.

Mr. Frost always took great pride in the fact that he had carried the entire first few editions of his paper to the post-office himself, and that, during his 37 years of ownership, the paper had never missed an issue and was always out on time.

In 1886, Mr. Frost became a resident of Plainfield, N. J., and from the first took great interest in the local affairs of that place. When the City authorities were considering a sewerage system for the

town, he took an active part in the work, serving, as a member of the Common Council, on the Committee on Streets and Sewers for five years. As Chairman of that Committee in 1893, he drafted the design for the sewerage system and devoted much thought, time, and labor to the project until its completion, in 1896.

After severing his connection with *Engineering News* in 1911, Mr. Frost retired from active business and devoted his time to study and travel. He was deeply interested in the study of genealogy and in geography and history, and he supplemented his reading by travel in many parts of the world. He had visited most of the countries of Europe and had made extensive trips to Egypt, South America, and in the United States. In 1914, he was planning a trip around the world, but gave it up on the outbreak of the European War.

Up to December 23d, 1916, Mr. Frost had enjoyed unusually good health. On that date, he suffered a stroke of paralysis and, although his condition improved for a time, he grew worse again, his death occurring at his home in Plainfield on March 15th, 1917.

On December 3d, 1868, he was married to Miss Louisa Hunt, a daughter of the late Edwin Hunt, of Chicago, Ill., and she, with four sons, survives him.

Mr. Frost was a thorough business man, with a capacity for the smallest as well as the largest detail. He was an optimist by nature, always looking on the bright side of things, his enthusiasm never lagging until his object was accomplished. Being deeply religious, he always took a keen interest in local church and charitable affairs, and was an active member of the Crescent Avenue Presbyterian Church of Plainfield.

Mr. Frost became a citizen of the United States in 1863, and was always afterward associated with the Republican Party. He was a member of the Canadian Society of Civil Engineers, and was also connected with a number of other scientific and technical organizations. At the time of his death, he was President of the Courier-News Publishing Company, of Plainfield, N. J., having purchased that paper in June, 1904.

Mr. Frost was elected an Associate of the American Society of Civil Engineers on January 4th, 1882.

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## PAPERS IN THIS NUMBER

"AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA." F. C. CARSTARPHEN. (To be presented May 2d, 1917).

"MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS." J. F. PARTRIDGE. (To be presented May 16th, 1917).

## PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

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"AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA." [illegible]

"MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS." [illegible] May 1917.

## PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

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MAY, 1917

Vol. XLIII. MAY, 1917. No. 5.

# AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS

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Vol. XLIII. MAY, 1917. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS



# OBSTRUCTION OF BRIDGE PIERS TO THE FLOW OF WATER

By Floyd A. Nagler, Jun. Am. Soc. C. E.

To be Presented September 5th, 1917.

## Synopsis.

**This paper presents the results of 256 experiments which were performed in a flume at the Argo Dam, Ann Arbor, Mich., during October-December, 1914.**

**The experiments were made on 34 different models of bridge piers, in order to determine the relative obstruction to the flow of water offered by piers of different designs.**

**The method of conducting the experiments is described, and the existing theories for this phenomenon are reviewed; however, in order to show the relative efficiencies of these piers, it was found necessary to discard these theories. The paper presents a unique and simple formula which can be applied to this problem, making possible the comparison of the different piers by the magnitude of their coefficients in this formula.**

**Tables I to VII* contain all the experimental data, good and bad.**

## The Problem.

**It is a well-known fact that the cross-sectional area of a stream of water cannot be either diminished or increased without causing energy losses, and hence losses in head, in the stream itself.**

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Tables I to VII are not printed with the paper, but have been filed in the Library of the Society, where they may be consulted by any one interested.

It follows, therefore, that when a bridge pier is constructed in a given stream section, such losses of head will be observed. For, at the nose of the pier, the stream is contracted in width, and, at the tail of the pier, it is suddenly enlarged to its original width.

Static head must first be available to accomplish the necessary increase in velocity demanded by the decrease in the stream section at the nose of the pier. Hence, there is observed the common phenomenon of "back-water", or the increase in the elevation of the water surface at the nose of the pier. It is also a well-known fact that all the static head, transformed into velocity head as the section converges, cannot be regained as the water diverges into the section of normal width at the tail of the pier. A loss of energy is thus suffered by the stream, in passing the bridge pier.

The losses, for convenience of these tests, are classified as follows:

(1) Losses which may be attributed to the change in section: These are made up of both surface and submerged losses, which are revealed by the turbulence and eddies at the nose and tail of the pier.

(2) Losses due to the friction of the water as it passes the wetted pier surface: These are relatively small when compared with those due to the change in section; they are so small, in fact, that, in most computations, they may be neglected.

It is at once apparent, on inspection of Fig. 8, that the shape of the nose and tail of the pier will have an appreciable effect on the magnitude of these losses. The pier, which will deflect the water at the nose, tangent to the sides of the pier, thus reducing to a minimum any further contraction of the stream section, should be most efficient at the up-stream end; and the pier with a tail which follows the diverging stream lines most closely, reducing the volume of the eddying water to a minimum, should be most efficient at the down-stream end.

Experimental data on the obstruction of bridge piers to the flow of water are exceedingly scarce; so scarce, in fact, that, in the case of the New York, Lackawanna and Western Railway *vs.* the New York, Lake Erie and Western Railway,* engineers varied in their computations of the back-water caused by the proposed bridge piers of the

* "On the Determination of the Flood Discharge of Rivers and of the Backwater Caused by Contractions", by William R. Hutton, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 211.

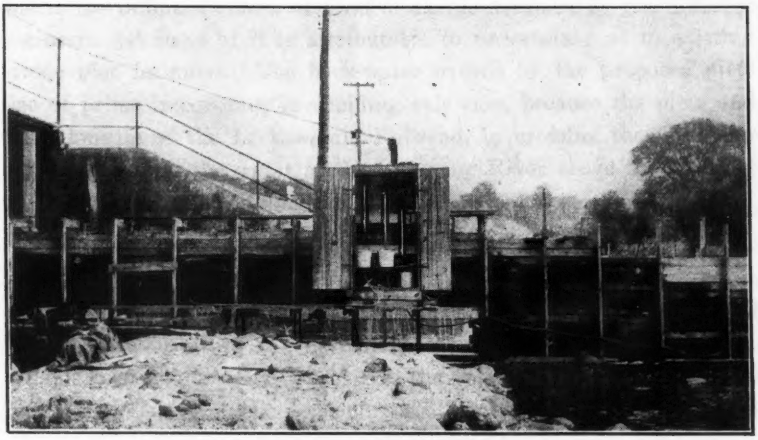


FIG. 1.—BOLTON'S FLUME.

FIG. 2.—BOLTON'S FLUME.

Lackawanna Railroad in the Chemung River, by values ranging from 0.568 to 4.3 ft. Although it is true that part of this discrepancy was due to the different values of flood discharge assumed by the different engineers, yet most of it is attributable to uncertainty as to existing bridge pier formulas. The back-water caused by the proposed piers was of prime importance in deciding this case, because the piers and embankments of the Lackawanna Railroad, in crossing the Chemung flats, might raise the water of the Chemung River above the grade of the Erie tracks in flood stages of the stream.

## Purpose of the Tests.

The purpose of these experiments was to determine the relative obstruction which different designs of bridge piers offer to the flow of water. This involved the derivation of an adequate formula, with reliable coefficients, which would give the amount of back-water caused by the insertion of a pier in the cross-section of the experimental flume.

## Description of the Flume.

The tests were made in a flume constructed at the Argo Spillway, Ann Arbor, Mich. A head of 13 ft. of water was available for the experiments. This flume was constructed by Frank L. Bolton, Jun. Am. Soc. C. E., and was used by him in making extensive experiments on the efficiency of trash racks of different designs.

Figs. 1 and 2 are reproduced from photographs of the flume taken in October, 1914. Fig. 3 is a plan and section of the flume.

The sides of the flume were made of 2-in. matched yellow pine; the flume averaged 2.138 ft. in width, and could carry a depth of water of 4 ft. The quantity of water entering the flume was regulated by lowering or raising the flood-gate in the spillway. As the water entered the flume at a very high velocity, it was first necessary to lower this velocity and distribute the varying velocities uniformly over the entire section of the flume. This was accomplished by a set of baffles, shown on Fig. 3, as Baffles No. 2. Each set of baffles consisted of a set of vertical bars and a set of horizontal bars about 5 in. apart. Fig. 4 shows that an excellent distribution of velocity was obtained, in this manner, at a section 15 ft. below the baffles and 6 ft. above the testing section.

The depth of water in the flume was controlled by vertical stop-boards just above the second set of baffles.

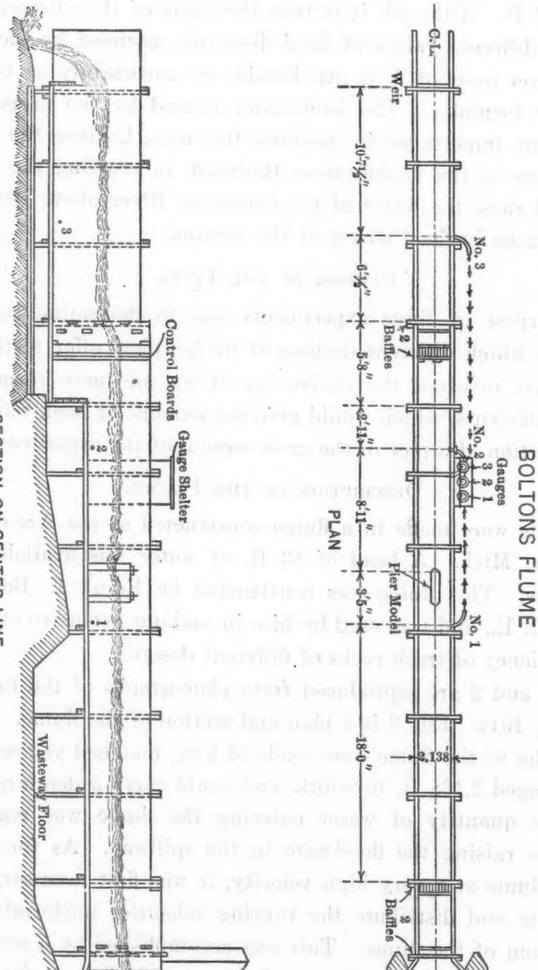
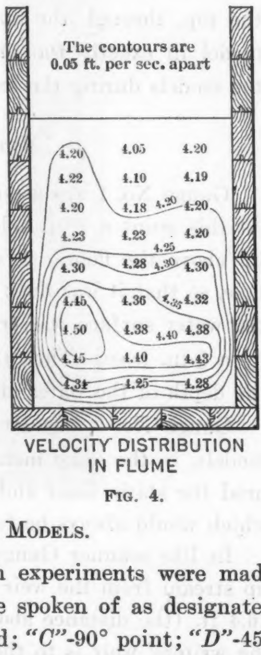

BOLTONS FLUME
PLAN
SECTION ON CENTER LINE
C.L.
Weir
No. 3
Baffles
Gauges
No. 4 3 2 1
Pier Model
No. 1
Baffles
10'-7½"
8'-8"
11'-11"
8'-11"
18'-0"
2.138
Control Boards
Gauge Shelter
Waterway Floor
Waste Gate

FIG. 3.

The second set of baffles, shown on Fig. 3 as Baffles No. 1, was a duplicate of the first. These were introduced into the flume in order to secure a uniform distribution of velocity in the channel approaching the weir.

The weir was at the end of the flume, with its crest 2.01 ft. above the bottom of the channel of approach. The sharp crest was a 3 by ½-in. bar of cold-rolled steel. The sides of the flume extended 4 ft. beyond the weir crest, in order that there should be no expansion of the nappe after leaving the crest. As is shown by Fig. 3, care was taken to build the bottom of the flume above maximum tail-water elevation, in order that it might be at all times apparent that the leakage from the flume was normal.

The hook-gauges used for all measurements of head are shown in Fig. 5. They were standard Gurley gauges, and measured the water surface elevation in the 12-qt. pails shown below the gauges. These pails served as stilling boxes, and were connected to their respective sections of the flume by ¾-in. rubber hose. All three gauges were placed in a small shelter house, and could be read by a single observer.




VELOCITY DISTRIBUTION IN FLUME

FIG. 4.

## Description of Bridge Pier Models.

The different shapes of piers with which experiments were made are shown in Fig. 8. They will hereafter be spoken of as designated on the plate: *"A"*-square end; *"B"*-half-round; *"C"*-90° point; *"D"*-45° point; *"E"*-thick double-curved point; *"F"*-single-curved point (convex); *"G"*-thin double-curved point. In such notation as *AA*, *BC*, *GD*, etc., the first letter indicates the form of nose, and the last letter the form of tail used in the experiment.

The models were of planed white pine. Each was 36 in. deep and 6 in. wide, although the gross length varied somewhat, according to the different designs. A center section 18 in. long was used for all piers; to this was attached the different forms of ends, in changing the

design. Figs. 5 and 6 show the pier models, how the piers were constructed, and the simple manner of attaching the ends to the center section.

A given design of bridge pier having been assembled; the pier was placed in the center of the flume at the point shown on Fig. 3. The pier was firmly clamped in place by two wooden clamps which spanned the flume above the water surface. The ¼-in. rods, which fastened the ends to the body of the pier, projected at both the bottom and the top of the model: at the bottom into a bushing in the floor of the flume; and at the top, through the clamp. This insured the testing of every pier model in exactly the same position, and prevented any vibration of the models during the test.

## Method of Experimentation.

Gauge No. 1 was attached to a point 3 ft. above the nose of the pier. At this point a ¾-in. hole was bored in the side of the flume, about 2 in. above the floor. A standard hose connection was fitted into this hole, so that it was flush with the sides of the flume. The elevation of the water surface was transmitted from this point to the stilling pail by a ¾-in. hose. Thus this gauge measured the static head, and also the depth of the water above the bridge pier models.

Gauge No. 2 was attached at a section 8 ft. below the tail of the models, in the same manner as Gauge No. 1. This gauge then measured the static head and depth of water at a distance below the pier which would always be farther down stream than the "standing wave."

In like manner Gauge No. 3 was attached at a point 10 ft. 7½ in. up stream from the weir crest. This distance is in the same ratio to 16.4 ft. (the distance above the weir used by Bazin), as the height of the writers' weir is to the height of the Bazin weir. In any case, the surface curve is so flat at this distance from the weir that the difference in discharge, from results which might have been obtained were it possible to measure the head farther up stream, is of negligible consideration in these tests.

The oscillations of the water surface in the flume were rendered very much less prominent by the rubber hose connection to the stilling wells. The water surface could easily be read to 0.001 ft. with the hook-gauges, and in many experiments to 0.0005 ft.

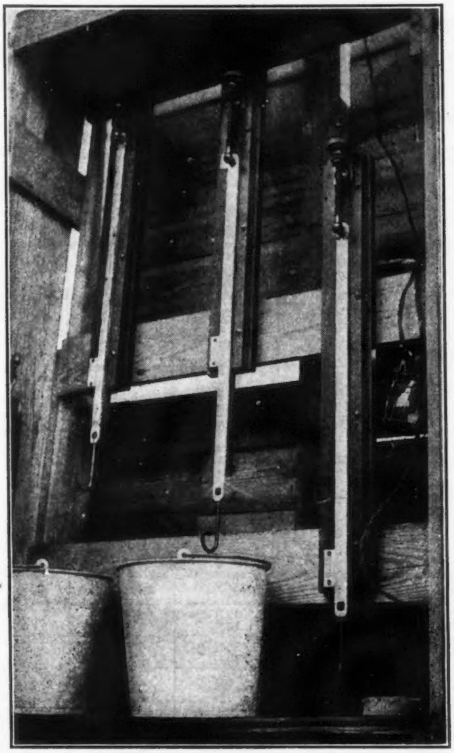


FIG. 5.—HOOK-GAUGES.

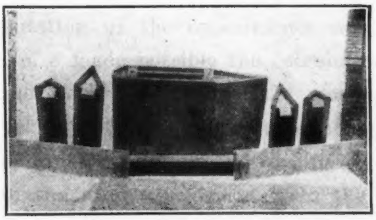


FIG. 6.—PIER MODELS.

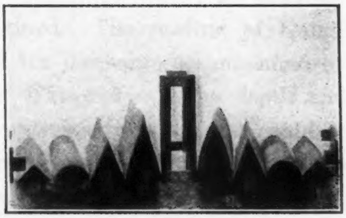


FIG. 7.—PIER MODELS.

The arithmetical mean of ten observations at a given velocity and depth of flow was used as the measured head for that experiment. The variation in these ten readings was remarkably small in all the runs. An average set of ten such observations is given in Table 1 as it appears in the field notes for Pier *BB*. The three gauges were read in rapid succession by a single observer, first No. 1, then No. 2, then No. 3.

COEFFICIENTS

RELATIVE NOSE EFFICIENCY

| "*A*" Tail | "*B*" Tail | "*C*" Tail | "*D*" Tail | "*E*" Tail | "*F*" Tail | "*G*" Tail |
|---|---|---|---|---|---|---|
| *AA* 0.861 | *AB* 0.866 | *AC* 0.862 | *AD* 0.873 | *AE* 0.875 | *AF* 0.863 | *AG* 0.863 |
| *CA* 0.894 | *CB* 0.905 | *CC* 0.893 | *CD* 0.902 | *CE* 0.907 | *CF* 0.905 | *CG* 0.906 |
| *FA* 0.903 | *DB* 0.918 | *DC* 0.912 | *DD* 0.916 | *DE* 0.931 | *DF* 0.929 | *BG* 0.927 |
| *EA* 0.908 | *BB* 0.923 | *BC* 0.915 | *BD* 0.923 | *BE* 0.935 | *BF* 0.931 | |
| *DA* 0.910 | *FB* 0.927 | *FC* 0.921 | *FD* 0.927 | *FE* 0.939 | | |
| *GA* 0.911 | | | | | | |
| *BA* 0.914 | | | | | | |

RELATIVE TAIL EFFICIENCY

| "*A*" Nose | "*B*" Nose | "*C*" Nose | "*D*" Nose | "*E*" Nose | "*F*" Nose | "*G*" Nose |
|---|---|---|---|---|---|---|
| *AA* 0.861 | *BA* 0.914 | *CC* 0.893 | *DA* 0.910 | *EA* 0.908 | *FA* 0.903 | *GA* 0.911 |
| *AC* 0.862 | *BC* 0.915 | *CA* 0.894 | *DC* 0.912 | | *FC* 0.921 | |
| *AF* 0.863 | *BB* 0.923 | *CD* 0.902 | *DD* 0.916 | | *FB* 0.927 | |
| *AG* 0.863 | *BD* 0.923 | *CB* 0.905 | *DB* 0.918 | | *FD* 0.927 | |
| *AB* 0.866 | *BG* 0.927 | *CF* 0.905 | *DF* 0.929 | | *FE* 0.939 | |
| *AD* 0.873 | *BF* 0.931 | *CG* 0.906 | *DE* 0.931 | | | |
| *AE* 0.875 | *BE* 0.935 | *CE* 0.907 | | | | |

FIG. 8.

From the ten gauge readings all information necessary for the computation of the experiments was obtained. The reading of Gauge No. 3 made possible the calculation of the discharge, as measured by the Bazin weir; from the reading of Gauge No. 2 the depth and velocity of the water at a section down stream from the pier could be obtained; and from the reading of Gauge No. 3 the depth and velocity of the water up stream from the pier could be computed; and from the readings of Gauges Nos. 1 and 2 combined, the back-water caused by the pier could be ascertained.

TABLE 1.—AVERAGE SET OF TEN OBSERVATIONS.

| Experiment. | Readings of Gauges, in Feet. | | | Time. |
|---|---|---|---|---|
| | Gauge No. 1. | Gauge No. 2. | Gauge No. 3. | |
| *BB*.5 | 0.871 | 0.833 | 0.395 | 9.13 A. M. |
| | 0.872 | 0.830 | 0.396 | |
| | 0.870 | 0.829 | 0.395 | |
| | 0.870 | 0.831 | 0.396 | |
| | 0.873 | 0.833 | 0.397 | |
| | 0.873 | 0.832 | 0.398 | |
| | 0.873 | 0.833 | 0.397 | |
| | 0.873 | 0.833 | 0.393 | |
| | 0.870 | 0.829 | 0.397 | |
| | 0.870 | 0.828 | 6.396 | |
| Mean. | 0.8715 | 0.8311 | 0.3960 | |

The elevations in Table I* were computed from levels run by Mr. Bolton, and re-checked by him after the gauges had been in place for several months.

In general, seven experiments were made on each pier, at a constant depth of water, varying the velocities in the flume from 1 to 4 ft. per sec. In addition to this, on Piers *AA, BD,* and *DB,* the depth of the water was varied for constant velocities. The different experiments on a single pier are designated in Tables I to VII by the numbers, 1, 2, 3, etc.

## Computations.

The discharge for a given run was read from the curves on Fig. 9. The upper curve gives the discharge of the weir as determined by Bazin's formula for a weir with a crest 2 ft. above the bottom of the approach channel. The lower curve was computed from the upper curve, by increasing the discharge 1% in order to compensate for the leakage of the flume between the weir and the bridge piers; thus, it gives the discharge at the bridge pier section.

The foregoing leakage correction was determined jointly with Mr. Bolton, by catching the leakage, in a definite period of time, from several of the 3-ft. sections of the flume. This was also checked by measurements with a small Ott current meter at the pier section, and also just above the weir.

* Tables I to VII are filed in the Library of the Society.

The width of the weir crest, and also the average width of the flume above it was 2.156 ft. The mean width of the flume at Gauges Nos. 1 and 2 was 2.138 ft.

Due to the limitations in the accuracy of the weir formulas, and also the fact that the experiments with the higher velocities gave heads as high as 2.1 ft. on the weir, it has been concluded that the absolute error in these tests may be as great as 3%, and the relative error as great as 2 per cent.

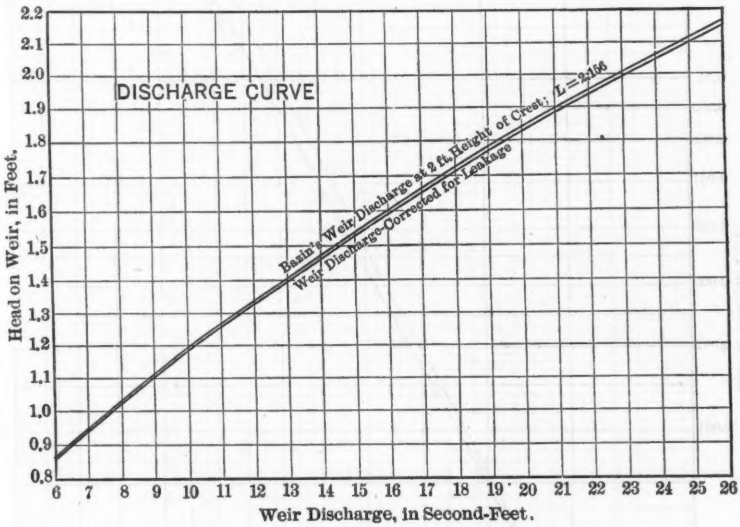



FIG. 9.

In all computations the pure friction losses of the pier models were neglected. This seemed to be permissible, as this loss amounts to less than 0.001 ft. for the highest velocity recorded in these tests. It was thus possible to make comparisons between piers differing in gross length due to changing the form of the nose or tail.

Mr. Bolton determined the friction losses of the flume itself, between Gauges Nos. 1 and 2, by a series of thirty-three experiments at different velocities and depths, with no obstruction in the flume. The pure loss due to friction was then computed by applying the principle of Bernouilli. The results were plotted as shown on Fig. 10. The mathematical mean line was determined for these points, and the equation of this line was found to be $H = 0.067\ V^{1.866}$.

After very careful study, it was found that the variation of the depth of the water in the flume, within the range of the experiments performed, seemed to produce so little variation in the value of the friction loss, that this loss in the flume for the varying velocities was

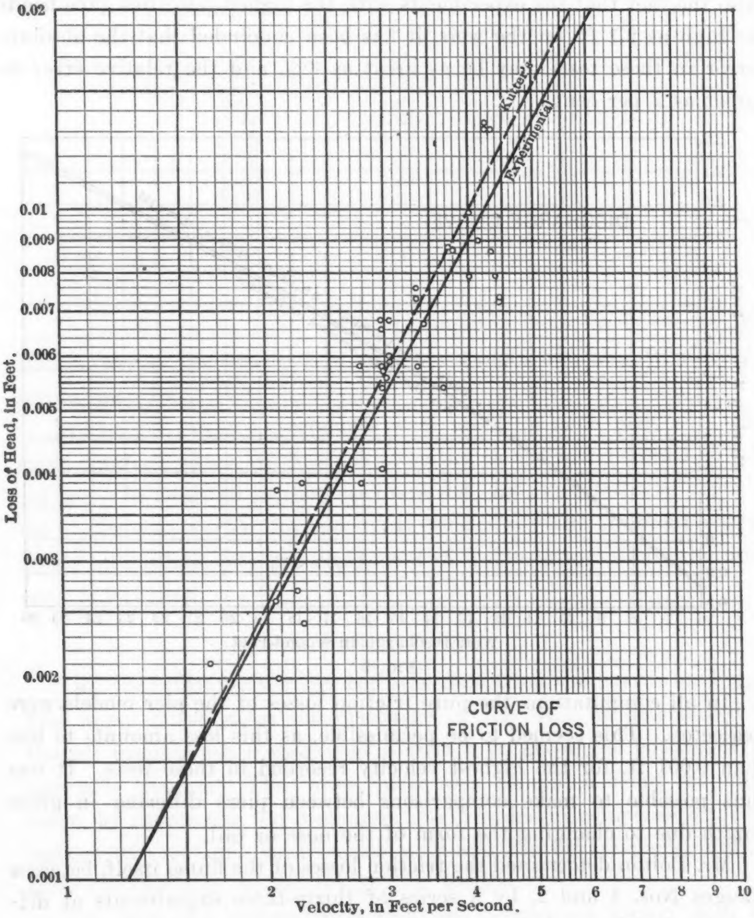



FIG. 10.

read from the mathematical mean line, regardless of the depth of water in the flume. The close proximity of this line to that obtained from Kutter's coefficient for a wooden channel, with water 3 ft. deep, is an excellent check on the accuracy of these experiments. A varia-

tion in depth of 0.5 ft. will cause a variation in the value of the friction loss of 0.001 ft. at the highest, and 0.00005 ft. at the lowest, velocities. In any case this is a negligible percentage of the total difference in water surface elevation caused by the insertion of any pier in the flume.

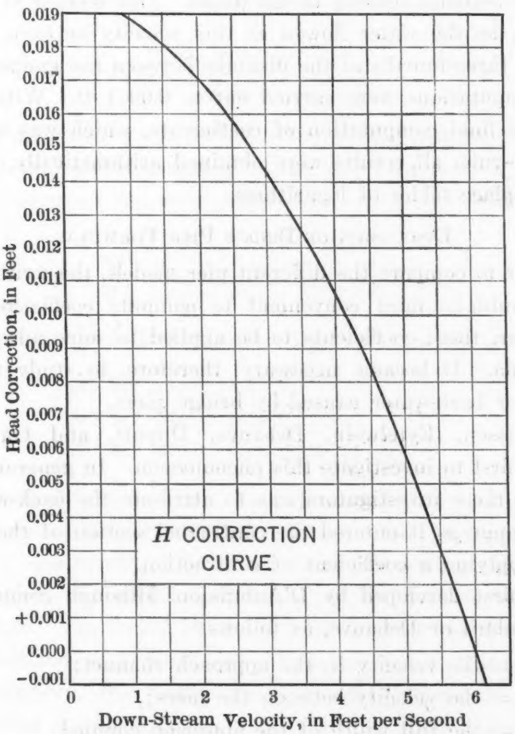



FIG. 11.

Now, if

$H_1$ = the depth of the water above the piers (Gauge No. 1);

$H_2$ = the depth of the water below the piers (Gauge No. 2);

$H_d$ = the difference in elevation between the bottom of the flume at Gauges Nos. 1 and 2;

and $H_f$ = the friction loss in the flume between Gauges Nos. 1 and 2; then $H_1 + H_d - H_2 - H_f$ = the difference in water surface elevation, or back-water, caused by the pier.

If $H_d - H_f = H_4$, then, as $H_d = 0.0195$ ft., the values of $H_4$ may be plotted against the velocity, as in Fig. 11.

The correction, as read from this curve, was applied to all values of $H_1 - H_2$ in the following manner: The value of $H_4$ for a given experiment was interpolated from the curve, at a point corresponding to the down-stream velocity of the flume. This seemed to be the most reasonable, as the water flowed at this velocity or even greater, for more than three-fourths of the distance between the gauges.

All computations were carried out to 0.0001 ft. With the exception of the final computation of coefficients, which was made with a 10-in. slide-rule, all results were obtained arithmetically, or with the use of six-place tables of logarithms.

## Discussion of Bridge Pier Formulas.

In order to compare the different pier models, the writer concluded that it would be most convenient to compute coefficients for each kind of pier, these coefficients to be applied to some adequate bridge pier formula. It became necessary, therefore, to study the existing formulas for back-water caused by bridge piers.

D'Aubuisson, Eytelwein, Debauve, Dupuit, and Gauthey were among the first to investigate this phenomenon. In general, the theory adopted by these investigators was to attribute the back-water to the velocity change as it entered the contracted section of the stream, in all cases applying a coefficient of contraction.

It was first developed by D'Aubuisson, although commonly attributed to Dubuit or Debauve, as follows:

Let $V_0$ = the velocity in the approach channel;
$V_2$ = the velocity between the piers;
$W$ = the full width of the approach channel;
$w$ = the contracted width at the pier section;
$h$ = the normal depth of the stream below the piers;
$y$ = the amount of back-water; and
$m$ = the coefficient of contraction.

Then, as
$$V_2 = \frac{V_0 W}{w}$$
$$y = \frac{V_2^2 - V_0^2}{2g} = \frac{V_0^2}{2g}\left(\frac{W^2}{w^2} - 1\right).$$

Or,
$$y = \frac{V_0^2}{m^2 2g}\left(\frac{W^2}{w^2} - 1\right).$$

Eytelwein, however, pointed out that $V_2$ is not equal to $\frac{V_0 W}{w}$, because, due to the crest contraction, the depth of the water between the piers is less than the depth ahead of them. He also argued that the coefficient of contraction should only be applied to the contracted width. He thus modified the foregoing formula until it assumed the form:

$$y = \frac{V_0^2}{2\,g}\left(\frac{W^2}{m^2\,w^2} - \frac{h^2}{(h+y)^2}\right)$$

This, in all probability, is the most correct formula which has been derived, taking the preceding theory as a basis.

Debauve expresses the same formula in terms of the discharge of the stream, $Q$.

$$y = \frac{Q^2}{2\,g}\left(\frac{1}{m^2\,w^2\,h^2} - \frac{1}{W^2\,(h+y)^2}\right).$$

Gauthey gives the following values for $m$ in this formula:

0.95 for acute angles,
0.90 for half-rounded heads,
0.70 for blunt, or square heads.

The preceding formulas have been disregarded by modern investigators, as the losses at the tail of the pier and the standing wave have been entirely neglected. This would tend to make $y$ less than its theoretical value, if observed at the head of the enlargement, where $y$ is a maximum. The losses at the tail of the pier are considered by many investigators to exceed those of contraction at the head of the pier. Furthermore, the results of these experiments, if substituted in the foregoing formulas, give a regularly decreasing variation in the value of $m$ for a single pier, within the range of from 1 to 4 ft. velocity, of more than 9 per cent. The values of $m$, computed from these tests, all lie between 1.07 and 1.28, and are thus much higher than those given by Gauthey. Certainly, in any case, a formula which requires a table of coefficients varying through a range of 9% is little better than no formula at all.

Weisbach* describes two experiments which he performed on a small round pier, 0.020 m. in diameter, in a channel 0.0280 m. wide.

* "Experimental Hydraulik."

He derived coefficients of 0.988 and 0.970 in the two runs which he made. These coefficients are to be applied in a formula based on the following theory:

In Fig. 12 if $H$ is equal to the difference in elevation between Gauges Nos. 1 and 2, he considers the discharge through the pier section to be equivalent to that through an orifice of a section, $AB$, and under a head, $H$, and over a weir the length of which is the net stream width at $A$, under a head, $H$. The sum of these two discharges should give the discharge past the pier.

Thus, if $W$ is equal to the width of the channel between the piers, and $D_2$ is equal to the depth of the water at Gauge No. 2, then

$$Q = C\,W\,\sqrt{2\,g}\left(\frac{2}{3}\,H^{\frac{3}{2}} + D_2\,H^{\frac{1}{2}}\right).$$

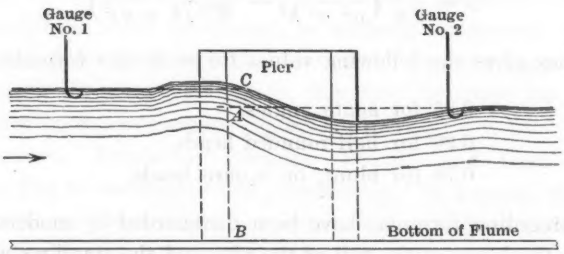



FIG. 12.

Mansfield Merriman, M. Am. Soc. C. E., adopts Weisbach's theory,* with some modification. In the first place, he corrects $H$ for the velocity in the approach channel, by adding to the measured difference in elevation, $H$, a quantity equal to 1 $\frac{V_1^2}{2\,g}$, where $V_1$ is the velocity of the stream. In the second place, he considers the weir portion of the theoretical discharge to be continuous over the entire width of the stream, and that the orifice discharges through the contracted section, only. Thus, if $B$ is equal to the width of the approach channel, and $b$ is the width at the pier section, then:

$$Q = c\,\sqrt{2\,g}\left[\frac{2}{3}\,B\left(H + \frac{V_1^2}{2\,g}\right)^{\frac{3}{2}} + b\,D_2\left(H + \frac{V_1^2}{2\,g}\right)^{\frac{1}{2}}\right].$$

* "Treatise on Hydraulics."

Assuming this theory to be the correct solution of the problem, coefficients were computed on the basis of Weisbach's equation, applying to $H$ a velocity of approach correction of $1 \frac{V_1^2}{2g}$. These may be found in Column 16 of Tables III to VII. These coefficients vary at least 20% in the case of each pier, being lower at the high velocities and as large as 1.200 at the lower velocities.

Again, the theory must be at fault, for a theory which gives as wide a variation of experimental coefficients as is shown in the tables is very little better than no formula at all. Attempts to reduce this variation in the coefficient by using different velocity of approach corrections did not produce favorable results. Likewise, using Merriman's modification of Weisbach's formula, the coefficients were found to vary throughout a still larger range, and, for the low velocities, they are practically equal to those tabulated in Column 16 of Tables III to VII. Merriman's assumption, that the weir discharges over the entire width of the stream, seemed to make conditions worse, rather than better. Certainly, there is little if any argument for assuming that the weir discharges over one section, and the orifice at another section, of the stream, and then adding the two to obtain the total discharge. A very fundamental principle of hydraulics is violated in so doing. Fig. 13 is a copy of the figure used by Mr. Merriman in demonstrating his formula.* Certainly, if the form of surface wave shown in the figure is the correct one, and the one which Mr. Merriman saw when watching water pass bridge piers, there may be more ground for the assumption that the weir discharges over the entire stream width.

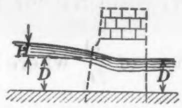



FIG. 13.

Weisbach, Gauthey, and others, however, have been very clear in their drawings in showing a rise in the water surface immediately ahead of the pier, and neither does it fall until it reaches the pier. This, certainly, is what has been observed in these experiments. Dupuit, also, in his theoretical investigation of the form of surface curve obtained by lateral contraction, arrived at the following conclusion:

"At the head of any contraction there must always be an elevation of the water surface above the normal; and at the head of any enlargement there must always be a depression of the water surface."

* "Treatise on Hydraulics," Ninth ed., p. 343.

Figs. 14 and 15 are reproduced from photographs of the bridge pier models in the flume with water flowing at a velocity of 4 ft. per sec. They show clearly the standing wave in front of the pier.

As the theory as a whole, however, does not coincide with these experiments, it was useless to dwell longer on this minor objection to Mr. Merriman's formula; the main error lies elsewhere.

If the writer had been required to calculate the loss in head through a sluice-gate with the form of section shown in Fig. 16, he might have computed this from the simple submerged orifice formula:

$$Q = C\,A\,\sqrt{2\,g\,H}.$$

Neglecting the friction of the concrete, the discharge would not differ whether $X$ were of concrete or of air. If of air, the form of the surface curve would be the same as that drawn in Fig. 12, and caused by the insertion of a bridge pier.

Furthermore, turning to the experimental data for the pier, $AA$, $H$, corrected for velocity of approach, $1\,\frac{V_1^2}{2\,g}$, was plotted as a function of $\frac{Q}{D_2}$ where $D_2$ is the down-stream depth of the water. The result is shown on Fig. 17. It was at once evident that $\frac{Q}{D} = f\,(H)$ was almost purely exponential, and that this exponent is approximately $\frac{1}{2}$. On investigation this was found to be true for the other piers also. These experiments thus seemed to indicate that the theory of considering the discharge past bridge piers as being that of a combined weir and orifice is false; and, furthermore, that the discharge might more correctly be represented as that through an orifice of a sectional area equal to the minimum stream width multiplied by the minimum stream depth, and under a head, $H$, equal to the back-water caused by the bridge pier (corrected for velocity of approach).

This theory is not new. Both Unwin and Frizell seem to have arrived at a similar conclusion in determining the discharge over a broad-crested weir. They found that

$$Q = L\,d\,\sqrt{2\,g\,(H - d)},$$

in which $L$ is the length of the weir; $d$ is the depth of water on the weir; and $H$ is the elevation of the water surface ahead of the weir, above the weir crest, as in Fig. 18.

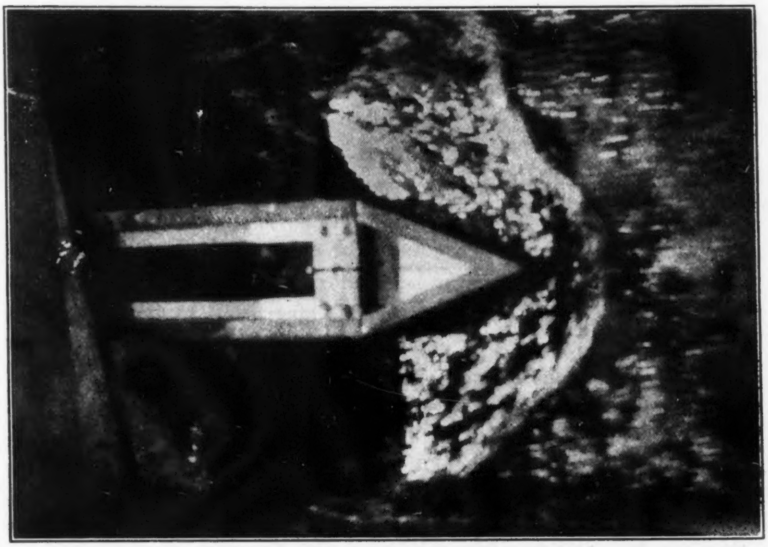


FIG. 14.—45° PIER NOSE.

FIG. 15.—HALF-ROUND PIER NOSE.

As $(H - d)$ is the difference in water surface elevation caused by the weir, the discharge has been computed as being that through an orifice at section, $B$, and under a net head, $h$. The crest and all hydraulic conditions in this case are very similar to the crest (obtained by the writer) due to the contraction of the channel by a bridge pier. In fact, Dupuit, in his theoretical investigation of this problem, states that the same resulting surface curve follows whether the contraction is from the sides or bottom of the stream. There may be some basis, then, for the assumption that, if this theory has been applied successfully to broad-crested weirs, it may as well be applied to bridge pier sections, where the surface curve is the same.

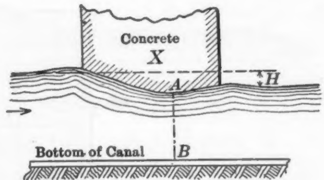



FIG. 16.

BRIDGE PIER FORMULA USED IN THE REDUCTION OF THESE EXPERIMENTS.

Referring to Fig. 19, the following base formula for discharge past a bridge pier—or back-water caused by bridge piers—was deduced as the result of these tests. In order to make comparison between the different pier models, the entire discharge in the stream is considered as passing through an orifice at the section, $A$.

If $A =$ the area of this submerged orifice,

$H =$ the head on that orifice, and

$Q =$ the discharge through that orifice,

then $Q = C A \sqrt{2 g H}$, in which $C$ is an empirical coefficient.

Now, in this case, $A = WD$, where $D$ equals the depth of water in Section $A$, and $W$ equals the width of the stream, exclusive of the bridge piers. Thus

$$Q = C W D \sqrt{2 g H}.$$

This is the base formula for these experiments, in its theoretical form, with no corrections. However, if the depth of water is measured at Gauge No. 2, designating this depth by $H_2$, then

$$D = H_2 - \alpha \frac{V_2^2}{2 g},$$

where $V_2 =$ velocity of retreat.

$H$ must also be corrected for the velocity in the channel of approach, thus

$$H = H_a + \beta \frac{V_1^2}{2 g}.$$

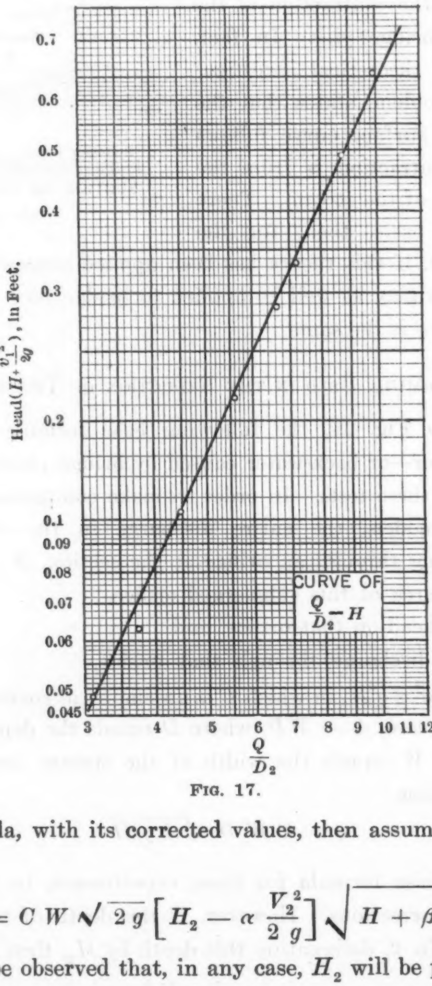



FIG. 17.

The formula, with its corrected values, then assumes the following form:

$$Q = C\, W \sqrt{2 g \left[ H_2 - \alpha \frac{V_2^2}{2 g} \right]} \sqrt{H + \beta \frac{V_1^2}{2 g}}$$

It should be observed that, in any case, $H_2$ will be practically equal to the depth of the stream before the erection of the bridge piers, and is equal to the depth of the water in the stream, at a point down stream, beyond the standing wave.

Taking all experiments into consideration, the best values of $\alpha$ and $\beta$ are as follows:

$$\alpha = 0.3 \text{ and } \beta = 1.8.$$

Using these values of $\alpha$ and $\beta$, the values of the coefficient, $C$, were computed for every run. These are tabulated in Column 19 of the tables. The mean value for each pier is given in Table 2.

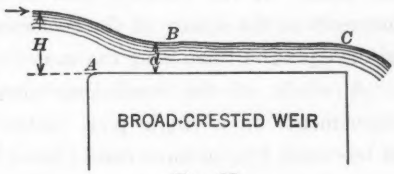


FIG. 18.

TABLE 2.—BRIDGE PIER COEFFICIENTS.*

$$Q = C\,W\sqrt{2\,g}\left[H_2 - 0.3\,\frac{V_2^{\,2}}{2\,g}\right]\sqrt{H + 1.8\,\frac{V_1^{\,2}}{2\,g}}$$

or,

$$H = \frac{Q^2}{2\,g\left[C\,W\left(H_2 - 0.3\,\frac{V_2^{\,2}}{2\,g}\right)\right]^2} - \frac{1.8\,V_1^{\,2}}{2\,g}.$$

| Type of pier. | Value of the coefficient, $C$. | Number of experiments of which coefficient is a mean. | Type of pier. | Value of the coefficient, $C$. | Number of experiments of which coefficient is a mean. |
|---|---|---|---|---|---|
| *AA*........ | 0.861 | 16 | *GA*........ | 0.911 | 4 |
| *AC*........ | 0.862 | 7 | *DC*........ | 0.912 | 7 |
| *AF*........ | 0.863 | 7 | *BA*........ | 0.914 | 7 |
| *AG*........ | 0.863 | 6 | *BC*........ | 0.915 | 7 |
| *AB*........ | 0.866 | 6 | *DD*........ | 0.916 | 7 |
| *AD*........ | 0.873 | 7 | *DB*........ | 0.918 | 15 |
| *AE*........ | 0.875 | 7 | *FC*........ | 0.921 | 7 |
| *CC*........ | 0.893 | 7 | *BB*........ | 0.923 | 7 |
| *CA*........ | 0.894 | 6 | *BD*........ | 0.923 | 17 |
| *CD*........ | 0.902 | 7 | *BG*........ | 0.927 | 6 |
| *FA*........ | 0.903 | 7 | *FB*........ | 0.927 | 7 |
| *CB*........ | 0.905 | 6 | *FD*........ | 0.927 | 7 |
| *CF*........ | 0.905 | 7 | *DF*........ | 0.929 | 7 |
| *CG*........ | 0.906 | 6 | *DE*........ | 0.931 | 7 |
| *CE*........ | 0.907 | 7 | *BF*........ | 0.931 | 7 |
| *EA*........ | 0.908 | 5 | *BE*........ | 0.935 | 7 |
| *DA*........ | 0.910 | 7 | *FE*........ | 0.939 | 7 |

* A complete tabulation of all data and computations may be found in the Library of the Society.

## DISCUSSION OF THE COEFFICIENTS AND THEIR LIMITATIONS.

The coefficients given in Table 2 are discharge coefficients, and are primarily valuable because they indicate the relative amount of obstruction offered by the different shapes of piers; that is, an increase of 1% in the discharge coefficient will cause a decrease of

about 7% in the total difference in water surface elevation above and below the pier, at a velocity of 4 ft. per sec. An inspection of the formula at once shows that the height of the back-water varies as the square of the velocity, also as the square of the discharge, and inversely as the square of the coefficient of contraction, all other values in the equation remaining the same.

A study of the coefficients obtained throughout the range of experiments on a single pier, shows variations from the mean value of less than 1%, in most cases; there being only ten experiments where the variation of the coefficients computed for any single experiment is greater than this. Far less variation than this could have been obtained by adopting different coefficients for the different pier shapes, but such a table of coefficients would be cumbersome, hence mean values of the coefficients were adopted ($\alpha = 0.3$ and $\beta = 1.8$) which gave the best results, considering the entire thirty-four different shapes of piers.

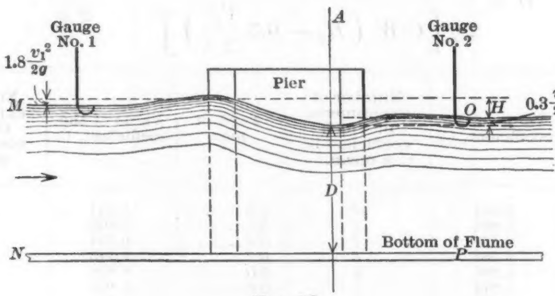



FIG. 19.

As the weir over which the discharge was measured was of limited capacity, these experiments cover a range in velocity of from 1 to 4 ft. per sec., only. Whether the coefficients for the formula will still have the same value for higher velocities is merely a matter of conjecture, although it would not be at all unreasonable to assume the same coefficients for slightly higher velocities of flow.

The formula is applicable to varying stream depths and hydraulic radii. This is shown by the coefficients computed for Piers *AA*, *BD*, and *DB*, on which tests were made through a very wide variation in the depth of the water in the flume.

As the velocity of retreat, for most problems, is only a very small amount greater than the velocity of approach, the computations may

be somewhat simplified by assuming them to be of the same value; hence, if $h$ is the head due to the velocity of approach, then

$$Q = C\,W \sqrt{2g}\,(H_2 - 0.3\,h)\,\sqrt{H + 1.8\,h}.$$

It has been claimed by some writers that the amount of back-water will vary with the relative velocities above and below the pier, the number of piers, the span of the bridge, the thickness of the piers, and the profile of the stream bottom.

The limitations in the size of the flume permitted tests to be made on only one particular width between piers. Whether these coefficients can be extended to other widths between piers is a matter of conjecture. The following are considerations which lead one to believe that this formula may be applicable to a stream of any width, and for any number of piers:

(1) Coefficients deduced by C. B. Stewart, Assoc. M. Am. Soc. C. E., on orifices 4 ft. square, have about the same value as those for small orifices. This principle may apply, as well, to the bridge pier section.

(2) Mr. Bolton's experiments on trash-racks prove that the loss in head due to racks of a given total area of obstruction, in the flume in which the tests were made, is practically the same, regardless of the spacing and width of the individual bars. Although these experiments are on a small scale, when compared with bridge piers, yet, in the absence of data on a larger scale, they add considerable weight in the solution of the bridge pier problem.

(3) The width of the stream is taken into consideration in this formula, in the large velocity of approach correction.

Weisbach's experiments, referred to herein, give values of coefficients which do not check with the values given here. His channel was so small, however, and his pier so nearly obstructed the entire flow of the water (leaving only 4 mm. on each side of the pier), that it does not seem at all unreasonable that his experiments should not check. It certainly has been a mistake to give these experiments as much prominence as they have received.

Working with the data afforded by the Lackawanna *vs.* Erie Case, mentioned previously, and assuming the flood discharge as computed

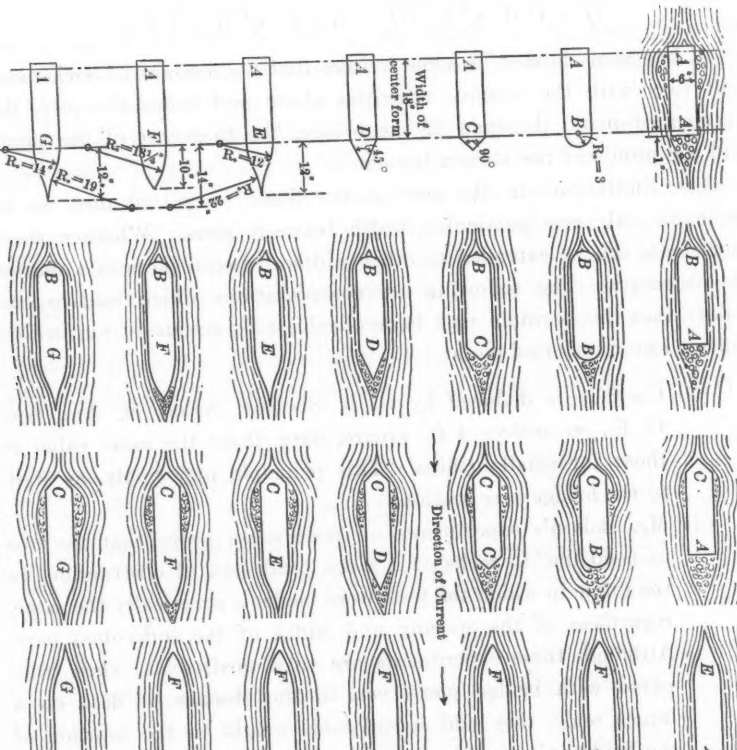



FIG. 20.

BRIDGE PIER MODELS

from Kutter's formula to be correct, a back-water of 2.30 ft. would have been caused with square-nose piers, or 1.95 ft. with the most efficient pier, if computed on the basis of the formula given herein. These results lie well inside the limits of the different results computed by the engineers who took part in this case, in fact, it is almost a mean of the two extreme results.

Fig. 20 shows how the coefficients vary with a given pier nose and different tails, and also the variation with a given tail and different noses.

The coefficients have been computed to the third decimal place, although the accuracy of the experimental work certainly does not warrant such refinement. This was done, however, in order to distinguish between the different designs of piers which have almost the same efficiency. On the other hand, any variation in the resulting coefficients, from that which one would expect from a general consideration of the effect produced by a given head or tail, occurs only in the third decimal place. In these particular cases of seeming inconsistency, carrying the results to the third decimal place may have been straining the accuracy of the actual tests. The almost general coincidence of results throughout the whole thirty-four different designs, even when considering the values of the coefficients carried to the third decimal place, speaks well for the relative accuracy of the experiments, in spite of a few inconsistencies.

## Conclusions.

These experiments have made the following contribution to our knowledge relating to the back-water caused by bridge piers:

(1) It has been demonstrated that the existing formulas for the discharge past bridge piers, and the back-water caused thereby, are erroneous.

(2) In working up the data provided by these experiments, a formula, which is substantiated by the results of 255 experiments, was developed. It is submitted as being far more adequate and simple than any of the existing formulas. Experimental data are not at hand, however, which will assure the accuracy of the formula, if applied to conditions differing widely from those in the writer's experimental flume.

(3) Coefficients for thirty-four different designs of bridge piers are submitted, for application in the writer's formula.

(4) The main value of these coefficients lies in the fact that they show the relative efficiencies of the different piers. This is shown in Fig. 20. The more remarkable conclusions may be summarized as follows:

I.—The best practical form of nose is either the half-round or the elliptical (of which Pier *F* is a modification). These shapes give much better efficiencies than the pointed nose.

II.—The best form of tail is the double curved form, like a fish tail (Pier *E*). The back-water is materially decreased by the adoption of such a tail. This form, however, is not practical; either the *D* or *B* modifications produce the second-best results. These are both practical shapes.

III.—The discharge coefficient may be increased from 1 to 3% by substituting an efficient tail for the square one. This means a decrease of from 7 to 25% in the amount of back-water at a stream velocity of 4 ft. per sec. This demonstrates clearly the importance of the shape of the tail of a pier.

IV.—The discharge coefficient can be increased from 5 to 6% by substituting a half-round nose for a square one. This means a decrease in the amount of back-water of from 35 to 45%, with a stream velocity of 4 ft. per sec.

V.—The decrease in the amount of back-water obtained by substituting a 90° point at the tail of the pier for the square end, is negligible; hence it does not warrant the extra expense of constructing such an end. The round tail is far superior to the 90° point, and is almost as efficient as the 45° pointed tail.

VI.—For piers of the same design, up stream and down stream, the half-round ends give the least amount of back-water.

## Acknowledgments.

For the facilities offered by the Argo Spillway and for the use of the water in the Argo Pond, the writer is indebted to the Eastern Michigan Edison Company and to Gardner S. Williams, M. Am. Soc. C. E., its Consulting Engineer. For the use of the flume

and valuable advice and assistance, and also considerable experimental data, the writer wishes to thank Frank L. Bolton, Jun. Am. Soc. C. E. The writer also wishes to acknowledge his indebtedness to Horace W. King, M. Am. Soc. C. E., Professor of Hydraulic Engineering at the University of Michigan, whose suggestions were invaluable in the conduction of these experiments.

## AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

# PAPERS AND DISCUSSIONS



## TESTS OF CONCRETE SPECIMENS IN SEA WATER, AT BOSTON NAVY YARD

### Discussion.*

By Messrs. Albert Larsen and W. Watters Pagon.

Albert Larsen,† Assoc. M. Am. Soc. C. E. (by letter).‡—In 1912 the Grand Trunk Railway System reconstructed a portion of the Back Cove Bridge, at Portland, Me., building two rest piers and re-capping the center pier. The center pier, built in 1892, was of granite ashlar, and was in such good condition that it is still used as the center pier, even with the increased loading. During the progress of the construction of the piers in 1912 Mr. Armour, Masonry Engineer, Mr. Durham, of the J. S. Metcalf Company, and the writer investigated the durability of concrete in sea water in order to determine its suitability for the new rest piers. Various concrete structures (including the test piles of the Aberthaw Construction Company), were visited in Boston Harbor, and from these observations it was finally decided to face the rest piers with granite ashlar, backed with concrete. The Grand Trunk Railway System contemplated future work in Portland Harbor, and therefore actual results of concrete placed in sea water were desirable. Ten 12 by 16-in. test piles, 16 ft. long, were made by various water-proofing methods, and immersed in sea water. Six piles were treated with integral water-proofing; two were plain, one of which was cured in the open for 3 months before being immersed; and two were treated with external water-proofing. A 1:2:4 mixture was used, with sand and crushed stone aggregates. The percentage of voids in the sand was 33. Mr. Larsen.

* Discussion of the paper by R. E. Bakenhus, M. Am. Soc. C. E., continued from April, 1917, *Proceedings*.

† Providence, R. I.

‡ Received by the Secretary, April 7th, 1917.

Mr. Larsen.

The proportions and different methods of water-proofing for the piles are stated in Table 11.

TABLE 11.

| Pile No. | Treatment. | Cubic feet of cement. | Cubic feet of stone. | Cubic feet of sand. | Gallons of water. |
|---|---|---|---|---|---|
| 1.... | Plain (cured in open, 3 months)............ | 5 | 20 | 10 | 25 |
| 2.... | Lye and alum (external)............ | 5 | 20 | 10 | 25 |
| 3.... | Plain............ | 5 | 20 | 10 | 25 |
| 4.... | Water glass (external)............ | 5 | 20 | 10 | 25 |
| 5.... | Water-proof cement ("*A*")............ | 5 | 20 | 10 | 25 |
| 6.... | Regular cement plus the water-proofing compound ("*B*")............ | 5 | 20 | 10 | 25 |
| 7.... | Regular cement plus the water-proofing compound ("*C*")............ | 5 | 20 | 10 | 29 |
| 8.... | Regular cement plus the water-proofing compound ("*D*")............ | 5 | 20 | 10 | 24 |
| 9.... | Regular cement plus the water-proofing compound ("*E*")............ | 5 | 20 | 10 | 23 |
| 10.... | Regular cement plus the water-proofing compound ("*F*")............ | 5 | 20 | 10 | 25 |

Water-proof cement was used in Pile No. 5, and the others contained "Knickerbocker" cement, either alone or in combination with water-proofing compound. A ½-yd. Smith mixer was used; each batch was turned 26 times, and the time was 1½ min. Portland City water was used.

The following results were obtained with the cement:

| | Neat. | Sand. |
|---|---|---|
| One-day test.................. | 390 lb. | ... |
| Seven-day test............... | 628 " | 318 lb. |
| Twenty-eight-day test......... | 735 " | 405 " |

The chemical analysis of the cement was as follows:

| | |
|---|---|
| $Si\ O_2$.......................... | 23.46 |
| $Fe\ O_3$.......................... | 9.35 |
| $Ca\ O$.......................... | 61.24 |
| $Mg\ O$.......................... | 3.59 |
| $SO_3$.......................... | 1.60 |
| Loss.......................... | 0.64 |

The piles were fastened to the pile trestle adjoining the spur-track leading to the coal pockets, and were made and immersed in the water on the following dates:

| | Moulded. | Placed. |
|---|---|---|
| No. 1.......... | October 10th, 1912. | January 23d, 1913. |
| 2.......... | " 3d, " | October 23d, 1912. |
| 3.......... | " 10th, " | " 24th, " |
| 4.......... | " 3d, " | " 24th, " |
| 5.......... | " 3d, " | " 18th, " |

FIG. 2.—SPECIMEN PILES TESTED IN SEA WATER.

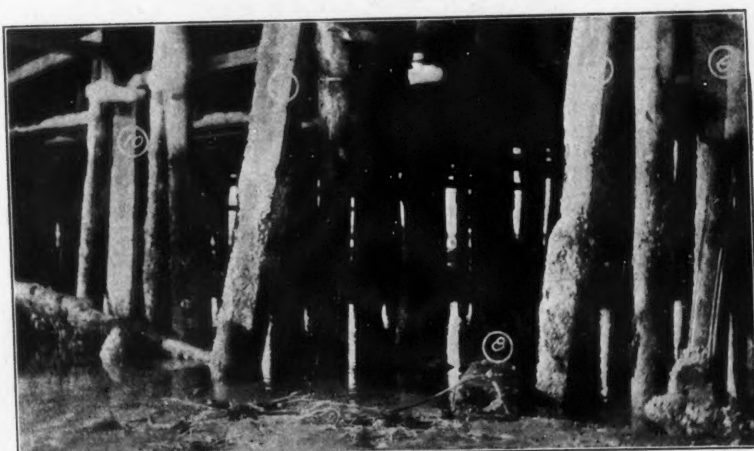


FIG. 3.—SPECIMEN PILES TESTED IN SEA WATER.

Mr. Larsen.

| | Moulded. | Placed. |
|---|---|---|
| No. 6 .......... | October 10th, 1912. | October 24th, 1912. |
| 7 .......... | " 4th, " | " 18th, " |
| 8 .......... | " 4th, " | " 18th, " |
| 9 .......... | " 4th, " | " 18th, " |
| 10 .......... | " 4th, " | " 18th, " |

The following is the formula for the Sylvester wash used in Pile No. 2: To 2 gal. of water add 1 lb. of concentrated lye and 5 lb. of alum, and mix until completely dissolved. This is a concentrated stock. When used, 1 pint of stock solution and 10 lb. of cement are mixed with enough water to make a mixture that will lather freely under the brush. Two coats are applied, the first as soon as the forms are removed (the surface must be wet), and the second as soon as the first is dry.

The formula for the water-glass or sodium silicate method of waterproofing (Albert Meyer) is as follows: After the forms are removed, grind with a carborundum stone any projections due to the concrete seeping through the joints between the boards. Keep the surface damp for 2 weeks after placing the concrete. Wash the surface thoroughly and allow it to dry. Mix a solution of 1 part water-glass (sodium silicate) 40° Baumé, with from 4 to 6 parts of water, total 5 to 7 parts, according to the density of the concrete surface treated. The denser the surface the weaker should be the solution. Apply the water-glass solution with a brush. After 4 hours and within 24 hours, wash off the surface with clear water. Again allow the surface to dry. When dry apply another coat of water-glass solution. After 4 hours and within 24 hours, again wash off the surface with clear water and allow to dry. Repeat this process for three or four coats, which should be sufficient to close the pores.

The water-glass which penetrates the pores comes in contact with the alkalies in the cement and concrete, and forms an insoluble hard material, causing the surface to become very hard to a depth of ⅛ to ½ in. according to the density of the concrete. The excess of sodium silicate which remains on the surface, not having come in contact with the alkalies, is soluble, and is easily washed off with water. The reason for washing off the surface between each coat and allowing the surface to dry is to obtain a more thorough penetration of the sodium silicate.

The tops of the concrete piers were treated by the water-glass method, and when examined on March 13th, 1917, were found to be in as good condition as when treated. The tops were finished with a 1:2 mortar with granite chips.

The piles were examined by the writer on March 13th, 1917, and photographs then taken are reproduced as Figs. 2 and 3.

The fungous growth was not removed from the piles.

Mr. Larsen.

The following indicates the condition of the piles:

No. 1 pile: Perfect condition.
No. 3 " : " "
No. 4 " : Nearly perfect condition.
No. 2 " : " " "
No. 5 " : Very marked deterioration at base of pile.
No. 7 " : " " " from low to high tide.
No. 9 " : " " " " " " " "
No. 6 " : Very decided deterioration from low to high tide; very bad condition.
No. 8 " : Very decided deterioration from low to high tide; very bad condition.
No. 10 " : Very decided deterioration from low to high tide; very bad condition.

The results are very interesting, and confirm in a general way the conclusion of Mr. Bakenhus that a 1:2:4 is better than a leaner mixture; that external water-proofing does not help when the concrete has been made as dense as possible; and that integral water-proofing has a detrimental effect on concrete.

From the writer's observations he would say that, once the concrete is affected by the action of the sea water, the rapidity of the disintegration is increased, as the water gets in between the stones and the mortar faster and thus breaks up the concrete. The writer thinks this may be overcome by facing the concrete with a dense mortar to a depth of from 1½ to 2 in. This may be done during the process of building, or afterward by the cement gun. It would be interesting to experiment with concrete made by either of these methods.

Mr. Pagon.

W. Watters Pagon,* M. Am. Soc. C. E. (by letter).†—The author has summarized the most valuable series of tests that have been made in the United States; tests that are on a par with the several series made in Germany, Norway, and Russia some years ago; and the writer feels that he should be accorded the sincere appreciation of the Profession.

The destructive action of sea water on concrete has become in recent years a very serious subject in all those cities on the Atlantic Coast north of New York City, and in corresponding climatic regions on other sea coasts. South of New York City, the writer is aware of only two cases of disintegration, one at Atlantic City‡ and the other in Chesapeake Bay, but both were special cases.

The writer's experience in a specific case on the Connecticut shore of Long Island Sound may be of interest, and is quoted briefly

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* Baltimore, Md.

† Received by the Secretary, April 13th, 1917.

‡ "Report on the Destructive Action of Sea Water on Concrete", reprinted from the *Monthly Journal* of The Engineers' Club of Baltimore.

here from a paper written by him, and now on file in the Society's Library. The concrete which suffered from the sea-water action consisted of the copings of a series of bridge piers which were faced with granite masonry below the coping. The facing extended up to mean high tide, but the copings, which extended from this point (Elevation 0) to Elevation + 1.5, were of 1:2:4 concrete without facing. Mr. Pagon.

Pier No. 4, which suffered most severely, was constructed on December 19th, 1913, during weather sufficiently cold to require the use of steam to warm the mixing water. After placing, the concrete was covered with a tarpaulin, stretched from one side of the cofferdam to the other, and cold steam was allowed to escape beneath this. During the week of January 19th to 24th, 1914, there was considerable freezing weather, accompanied by high tides. The result was the formation of a thick ice coating over the concrete surfaces. When this had melted, on January 26th, it was found that the surface of Pier No. 4 had been loosened, and large pieces of scale could be removed. Within a week the mortar between the stones of the gravel had disintegrated and fallen out, leaving the stones projecting. The loose portions were removed with a hammer, and behind them the concrete was perfectly sound. At the end of two weeks, the ice coating formed again, and when this had disappeared the erosion was found to have increased greatly.

The concrete used throughout the work was of a wet "sloppy" consistency. The forms were tight, so that there were no pockets. Immediately after removing the forms, the surfaces were rubbed with carborundum blocks. Cow Bay sand was used, and a mechanical analysis showed it to be too fine; there was also some clay. Analysis of the cement showed it to be excellent. Later, a coarser sand was used, with the results described hereinafter.

In the paper quoted, the writer discussed in detail the various possible causes of this disintegration, and he refers interested readers thereto. He felt that undoubtedly the fineness of the sand was one of the factors causing disintegration, and, therefore, drew up a specification for the sand which was used in all later work, under which, for acceptance, not more than 70% should pass the No. 20 sieve. Long Island sand testing as low as 40% could be obtained from one dealer, but it was found that sand which tested below 60% was too coarse for reinforced concrete work, where a smooth, easy flowing concrete is necessary. To avoid delay in obtaining sand, 70% was adopted as a criterion, and immediately a noticeable change in the strength of the concrete was shown in all the field tests, in the time of setting of the concrete, and also by observation of the concrete itself.

In the writer's opinion, however, the sand was by no means the governing factor. From the detailed description which is given in

Mr. Pagon. the paper quoted, it will be seen that the first failure of the surface skin on the concrete always took place where a pebble of the gravel aggregate was close to the surface, and where, therefore, the surface skin had small adhesion to the backing. After this area of skin had peeled off, it was a simple matter for the water, working its way along the surface of the pebble, to force off (by freezing) the surface skin between pebbles. When the skin was gone, the porosity of the concrete, due to the fineness of the sand, caused the disintegration of this mortar, as previously described. Photographs of the concrete just after the beginning of disintegration revealed innumerable little "pock-marks" where the pebbles showed through the broken skin.

In order to test this theory, three specimens were made for the writer by the contractors, McHarg-Barton Company. These were 4 by 4-in. by 10 ft., reinforced axially with a $\frac{3}{8}$-in. round bar, hooked at the upper end. The mixtures were as follows:

Specimen No. 1..................... 1 : 2 : 0
" No. 2..................... 1 : 2 : 4
" No. 3..................... 1 : $1\frac{1}{2}$ : 3

These were removed from the forms in 24 hours (in order to simulate as nearly as possible the temperature and greenness conditions of the pier coping). No. 2 was cracked in handling in one place; No. 3 in three places. When 3 days old the specimens were hung in the river so as to extend above high tide and below low tide.

After the first tide, they were all covered with a white coating, except on the troweled face. At the end of 5 weeks, while they were being shifted to another place, they were accidentally lost. No. 1, however, was in perfect condition; Nos. 2 and 3 were rather badly eaten away, especially at the cracks, where, in fact, No. 2 was eaten away so completely as to expose the rod.

The theory having proved satisfactory, so far as the test went, it was decided to cut away the face of the coping and replace it with 1 : 2 mortar made of sand bought under the new specification. This was done on April 4th, 1914, the mortar being mixed to such a consistency that it could be poured and stirred like thick cream. The forms were left on for a couple of weeks to protect the concrete from the sea water as far as possible. When they were removed, the mortar was in good condition, although there were numerous instances of bubbles having formed while it was being handled and placed. The surface was not rubbed for fear of causing injury to the protecting skin.

When examined 2 years later, this mortar facing was in perfect condition, although it had passed through two winters. This would seem to confirm the theory, in so far as one test can confirm it.

If other members of the Society have had similar experiences, it would certainly be valuable to hear from them, and the writer's only regret has been that in the Aberthaw tests, described by Mr. Bakenhus, none of the specimens was of mortar only. The writer feels strongly that the relatively good condition of the 1 : 1 : 2 specimens is due not only to the greater percentage of cement to aggregate, but also to the greater percentage of cement and sand combined to stone, or, in other words, that a 1 : 1 : 2 mixture approaches more nearly to a mortar. Mr. Pagon.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## MULTIPLE-ARCH DAMS ON RUSH CREEK, CALIFORNIA

### Discussion.*

By Messrs. F. O. Blackwell and A. D. Flinn.

F. O. Blackwell,† M. Am. Soc. C. E.—This paper is a valuable contribution to the theory of multiple-arch dam design, particularly as to the economical spacing of the piers and the thickness of the arch. The dam sites are on small drainage areas, high in the mountains, where labor and materials are very expensive. It has been stated‡ that the cement for these dams cost $7.50 per bbl. Mr. Blackwell.

A very conservative design might have been prohibitive in cost, and probably the region is uninhabited and no damage to life or property would result in case of failure.

The sections appear to be very thin for a 40-ft. span, and the reinforcement is extremely light. The use of more concrete would have added much to the ability of the structure to resist unexpected floods, ice thrust, changes of temperature, etc.

It is always desirable, too, to have a good margin over theoretical quantities in order to allow for poor material and workmanship, which will occasionally be found, no matter how careful the engineeer may be.

A fair comparison between this dam and other types is difficult to make, as the standards are quite different, but the multiple-arch appears to be cheaper than any other kind under the conditions which existed at the location, except possibly a rock fill with a thin vertical

* This discussion (of the paper by L. R. Jorgensen, M. Am. Soc. C. E., published in March, 1917, *Proceedings*, and presented at the meeting of April 18th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ In an article in *Engineering News*.

Mr. Blackwell. core-wall. The author states that there was loose rock but no earth at the sites.

The usual type of solid, retaining-section, gravity dam would have required about four times as much concrete as was used in the dam as built, but from 25 to 40% of cheap rock might have been laid in thick concrete sections, and concrete containing less cement might have been used. The form work and cost of forms and placing would also be less in a large mass.

Including engineering and overhead expenses, the multiple-arch dam probably cost about $30 per cu. yd., so that a gravity dam would have to be built for $7.50 per cu. yd. in order to equal it in cost.

In an accessible location, and with concrete rubble masonry at from $3 to $4 per cu. yd., the solid dam, in the speaker's experience, has always figured out to be cheaper than any multiple-arch or reinforced design.

Compared with a reinforced concrete slab dam, the multiple-arch seems to have the advantage in cost. The piers would have to be twice as many, the deck thicker, and the reinforcing metal in the deck about ten times as great.

One of the principal objections to a multiple-arch is the fact that the sections are all dependent on each other for stability. If one pier should settle, the arches on each side of it would break, and, as there is nothing to take the thrust of the remaining arches, they would push over their piers, and the whole dam would collapse. The struts between the piers could not be relied on to save the dam from complete failure, as the movement of any pier would throw them out of line and cause a resultant thrust, tending to push the pier in the direction in which it had already started to move. Rigid foundations, such as the author states exist in this case, are essential.

The Great Western Power Company started to build a multiple-arch dam at Great Meadows, on the Feather River, California, but abandoned it in favor of a rock fill with an earth face sluiced into place on the up-stream side to make it water-tight and cover the bottom for a considerable distance up stream. The lava foundations, on being opened up, were found to contain mud seams, which might have moved under the piers. As a general principle, a dam should be no better than its foundations. On a yielding bottom a dam of a flexible type is required.

There is one important matter that Mr. Jorgensen does not mention: At an altitude of 9 000 ft. in the Sierras it must be very cold at times, and thick ice must form in the reservoir. The thin arch is not much protection against cold, ice must form to a considerable thickness on the face of the dam, and such cracks as develop will be filled with ice. There may also be ice thrust against the dam, which could have serious results. With a mass of ice frozen fast to the arches, the inclina-

tion of the face would not relieve this pressure. Floating masses of ice may also drift against the arches, thus causing concentrated pressures on parts of the dam, and the arches are not designed to stand unbalanced pressures. One advantage of the ordinary form of masonry dam is the weight of its large mass, which will stand considerable impact without damage. Mr. Blackwell.

The spillway does not look very large, and would be likely to be choked with floating ice, in which case the water would go over the top of the arches. If this occurred, and the water carried ice with it, the ice would fall on the struts between the piers and might damage them. A deck on top of the dam would carry the water so that it would fall below the dam and not injure the struts and the foundations. It would also strengthen the top of the arch against ice thrust and would tie the piers together better.

A pressure of nearly 23 tons per sq. ft. seems to be extreme for a practically unreinforced and slender pier 90 ft. high.

The coefficient of friction against sliding on the foundations is 0.80 as compared with 0.65 in most solid dam designs. If the rock were smooth or horizontally stratified this might be unsafe.

It would be very interesting to hear from the author as to whether there has been any leakage through the arches, or whether any trouble has been experienced from ice.

A. D. Flinn,* M. Am. Soc. C. E.—Minimizing the quantity of masonry to be used in a dam naturally occurs to any one who has given much thought to the design of large masonry dams. The idea of accomplishing this by arches and buttresses seems to have occurred to engineers in different parts of the world many years ago, for such dams have been built in India and Australia, as well as in America. They have sometimes been called buttressed dams. There appear to be three important varieties: One, of which the Meeralum Dam in India is an example, has the arches vertical, with the buttresses sloping on their down-stream sides or faces; a second variety is similar, but has the spandrel spaces filled with masonry; and the third variety, to which the dams described in the paper belong, has the arches inclined, with the down-stream faces of the buttresses vertical, or nearly so. Mr. Flinn.

Mr. Blackwell has referred to the disadvantages of thin sections in such structures. The speaker would like to emphasize that point, on the basis of some experiences with concrete within the last few years. Possibly, some other engineers have also had experiences showing that Portland cement concrete under certain conditions disintegrates when exposed to weather, and particularly when exposed to weather and water together. The cause for such disintegration is

* New York City.

Mr. Flinn. not yet fully known, but it seems to be due in part to abuse of the concrete (or rather of the cement in it) by the use of excessive quantities of water in mixing, and leaving it in the concrete when it is finally placed in the forms. The speaker knows of one or two cases in which disintegration, apparently due to this cause, has gone on rapidly. Therefore, it is quite evident that a structure like a dam, with walls as thin as 12 to 24 in., would not long resist such an attack, if it should set in.

Another question—one of economy—has occurred to the speaker. In the East, dam sites are usually overlaid with glacial drift or other earthy deposit to considerable depths. Rarely does one find the nearly exposed rock foundations with which the West is favored in many places. It would seem that digging through a considerable depth of earth for the arches and the buttresses would be a handicap to the multiple-arch dam, in comparison with the "solid" masonry dam.

Another thought which occurs to one in looking at dams of this design, is along a line on which American engineers have not directed very much attention until within the past 2 or 3 years, and that is, the vulnerability of structures to attack either by malicious persons or by an enemy. It seems to the speaker that a "hollow" dam might be easily wrecked, if not guarded, in time of war, strike, or riot, and that a person of malicious intent, in the case of any such trouble, could readily do a structure of this kind great injury in a few hours, or even in a briefer time. One of the important considerations in providing for the guarding of a structure is to have it so arranged or so strong that it can be easily protected, and by the minimum number of troops.

In comparing the three forms of multiple-arch dams which have been mentioned, the question naturally arises: What are the relative advantages between the arch set vertically with the buttresses sloping on the down-stream side, and the type of dam described in the paper, with the arches sloping and the buttresses having vertical faces on the down-stream side? Is there a distinct advantage in either case, or is the difference solely due to an endeavor to secure patent rights, or some other commercial advantage? If any variety of multiple-arch dam has advantages over the others, will not the author or some other engineer who has devoted special study to this type, state these advantages? The speaker is interested as to the readiness or difficulty of making a dam of this kind, together with its foundations, sufficiently water-tight, and what standard of water-tightness is accepted. Would the leakage permitted be greater than that from a well-built "solid" masonry dam? As time goes by, do the agencies which make for greater water-tightness prevail over those which tend to produce leaks?

AMERICAN SOCIETY OF CIVIL ENGINEERS
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# PAPERS AND DISCUSSIONS



## CEMENT JOINTS FOR CAST-IRON WATER MAINS

### Discussion.*

BY MESSRS. HARRY Y. CARSON, F. M. RANDLETT, WALTER PEARL, AND H. G. MOULTON.

HARRY Y. CARSON,† JUN. AM. SOC. C. E. (by letter).‡—It would be a valuable addition to engineering literature if some one were to compile a comprehensive and authoritative paper or thesis on joints for cast-iron pipe. Much has appeared in the technical press, as well as before technical societies, with reference to the leakage of water and gas from underground mains, but, obviously, not enough has yet been done to bring about a general improvement in the prevention of these large percentages of wasted water and gas. This paper is a valuable contribution to such literature as now exists. Mr. Carson.

The causes for the normal leakage at joints in mains may be classified as follows:

1.—Contraction and expansion;
2.—Unequal settlement;
3.—Vibration and shock.

A fourth cause may be corrosion, but a very able paper entitled "External Corrosion of Cast-Iron Pipe"‖, by Marshall R. Pugh, M. Am. Soc. C. E., points out that deterioration in cast-iron pipe, as compared to other materials, is practically nil; that the oldest cast-

* This discussion (of the paper by Clark H. Shaw, Assoc. M. Am. Soc. C. E., published in March, 1917, *Proceedings*, and presented at the meeting of April 18th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ Received by the Secretary, April 9th, 1917.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 806.

Mr. Carson. iron pipe, flanged, and in 1-m. lengths, put into service more than 250 years ago has shown no appreciable loss in wall thickness or strength, and that: "experience has not been sufficiently long to establish just what its life is." The chief consideration for cast-iron pipe, therefore, may be said to center about the type of joint best suited to prevent waste through leakage.

Contraction and expansion exist in all mains, and vary in direct proportion to changes in the temperature of the pipe line. The movement caused thereby, though slight, and scarcely ever exceeding 2 in. per 1 000 ft., is absolutely irresistible, and its effect must be taken care of properly, or disaster is sure to result. No material is of sufficient strength to resist its power if rigidly maintained against it. Contraction and expansion produce by far the greatest proportion of normal leakage in mains. Because of its inelasticity and fragility, cement, as a material for jointing cast-iron pipe, has always been questioned. This in spite of the fact that it is very cheap.

The author makes the following statement:

> "Long Beach now has 60 miles of cast-iron water mains, * * * laid with joints of this [cement] type. All these pipes are under pressures ranging from 40 to 80 lb. per sq. in., and are giving perfect satisfaction."

He presents no data, however, by which the satisfactory performance of the line may be measured, as he has not submitted any information as to tests made for leakage or figures as to its quantity. Manifestly, without this information it is difficult to see how the line can be stated to be perfectly satisfactory.

The principal advantage of bell and spigot pipe when packed with lead lies in its ability to take care of the always existent contraction and expansion at its joints, without fracture. Cement joints may reduce the number of joints actually leaking, but rigidity is increased to such an extent that there may result an increasing number of fractures at those points. The net leakage with cement joints, however, is probably not materially different from lead.

Flanged jointed pipe, in order to withstand expansion and contraction, must have expansion joints at very frequent intervals.

Regardless of the kind of jointing material used, the unequal settlement of a pipe line may cause normal leakage as well as breakage, with extraordinary leakage. Damage ensues from settlement almost in direct proportion to the relative rigidity of the joint. Flanged pipe is particularly unsuited to conditions where unequal settlement prevails, as is the case in the down-town or business districts of cities, where the streets are so frequently torn up.

Vibration and shock are exceedingly deleterious to joints which are rigid, and here again the pipe line suffers in direct proportion

FIG. 10.—SUSPENDED LINE OF PIPES WITH METAL-TO-METAL JOINTS.

to its rigidity. The general results are similar to those caused by unequal settlement. Moreover, main leakage will increase as the system gets older, on account of the loosening of the joints, through contraction and expansion, settlement, and vibration. Mr. Carson.

A pipe joint in which packing materials of every character are entirely eliminated, so as to have a ground metal-to-metal connection, should give the most desirable results of all. Joints of this construction have now been in successful service for more than 15 years; and their more general adoption will proceed from the excellent results which have been obtained from their use for cast-iron mains. In joints of this type the bell and spigot ends of the pipe are machined at slightly different tapers, so that, when entered in a tight manner, the line may settle without leakage. Such a pipe line is capable of taking a rough contour over the trench bottom, or of assuming a comparatively small radius on any change of alignment.

Without the slightest leak whatever, pipe and fittings made with the machine flexible joints have been used successfully to convey hot water and fluids, such as steam, where the maximum expansion and contraction would be expected.

Furthermore, cast-iron pipe in which all jointing material is eliminated gives an ideal line. It cannot fail by subsequent deterioration of the joining material. For instance, there are many alkaline soils* where cement is quickly attacked by chemical action, which, however, does not corrode the pipe itself.† Moreover, such materials as lead, oakum, and cement, by virtue of their resistance to stray electric currents, are known to increase the damage to cast-iron mains, that sometimes follows the improper return of electric currents from trolley car systems.

In so far as settlement and vibration or shock become effective factors in causing leakage from cement and lead joints, or from flanged joints due to breakage in the latter, no leakage has resulted from proper metal-to-metal joints, and records have repeatedly shown that such joints have withstood successfully and without failure the most severe conditions known in practice.

Fig. 10 illustrates in a striking manner the efficiency of joints of this type. The photograph from which this half tone was made was taken in April, 1914, and shows a line that was laid across the top of a dam at the plant of the Semet-Solvay Company, at Holt, Ala. The dam was washed out by a freshet. The pipe settled as shown in the photograph, but all joints remained intact. The pipe line continued to give uninterrupted service while the dam was being rebuilt, supplying water at a pressure of about 50 lb. per sq. in. The sixteen

* Investigations by Bureau of Standards, U. S. Dept. of Commerce (*Metal Worker*, October 6th, 1916, p. 136).

† *Technologic Paper No. 25*, Bureau of Standards, U. S. Dept. of Commerce.

Mr. Carson. lengths of pipe affected, of which twelve lengths were entirely without support, showed a maximum settlement or deflection of 46 in. at the center of the line.

Mr. Randlett. F. M. RANDLETT,* ASSOC. M. AM. SOC. C. E. (by letter).†—This paper is of added interest to the writer as the Water Department of Portland, Ore., has been experimenting with cement joints for the past year.

During 1916, there were laid 1 910 ft. of 8-in. and 362 ft. of 12-in. cast-iron pipe with cement joints, with the most satisfactory results.

For the purpose of investigation, and to illustrate the manner of using the materials, three lines of 8-in. cast-iron pipe were laid, according to the specifications of the New England Water Works Association, as follows:

Line No. 1. Neat cement joints; 3.5 lb. per joint; some waste; cement joint about 3 in. deep.

Line No. 2. Leadite joints; 4 lb. per joint; 2 in. of leadite.

Line No. 3. Pig lead joints, caulked; 13.11 lb. per joint; 2 in. of lead.

Each line consisted of eight full lengths, with the four center lengths arranged so that their supports could be removed.

All were laid approximately level, and were plugged and subjected to 85 lb. internal water pressure continuously after completing and setting the joints.

The two end lengths of each line were braced securely, and were supported and weighted down with fifteen lengths of 8-in. cast-iron pipe laid across the three lines on each end.

Batter boards were set up over each of the five joints, and a center-punch mark was put on the pipe under the center line mark for each line on the batter boards. A flat space was filed on each length before being center-punched, and all measurements were taken later between the pipe and the batter boards at these points.

Several observations were taken from time to time, but Tables 3 and 4 show the results of the tests as completed. Line No. 1 was loaded with 200 lb., placed at the center of each length after the supports were removed.

Line No. 2 developed leaks at once, but not until one or two of the joints had pulled out considerably were they in such a condition that recaulking would not have been sufficient to stop the leaking entirely.

Line No. 3 developed leaks at several joints, varying from a fine stream to a slow drip. All these leaks had practically stopped at the time of the last observation.

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* Portland, Ore.

† Received by the Secretary, April 16th, 1917.

Mr. Randlett.

TABLE 3.

| Pipe. | LINE No. 1. | | LINE No. 2. | | LINE No. 3. | | Remarks. |
|---|---|---|---|---|---|---|---|
| | No. | Weight, in pounds. | No. | Weight, in pounds. | No. | Weight, in pounds. | |
| *a*......... | 2505 | 553 | 1826 | 550 | 1251 | 580 | |
| *b*......... | 2101 | 593 | 1227 | 571 | 1270 | 560 | |
| *c*......... | 2135 | 563 | 1822 | 566 | 1217 | 570 | Suspended lengths. |
| *d*......... | 533 | 565 | 1300 | 570 | 1329 | 560 | Suspended lengths. |
| *e*......... | 2439 | 574 | 1267 | 562 | 1775 | 582 | Suspended lengths. |
| *f*......... | 2526 | 566 | 455 | 560 | 903 | 577 | Suspended lengths. |
| *g*......... | 1751 | 570 | 2390 | 567 | 884 | 579 | |
| *h*......... | 2862 | 566 | 2220 | 570 | 1753 | 581 | |
| Averages.. | ...... | 567.5 | ...... | 564.25 | ...... | 573.625 | |

TABLE 4.—JOINT TESTS.—MEASUREMENTS ON OBSERVATION POINTS. The First and Last Measurement in Each Observation is on the Fixed Pipe at Each End of Each Line.

Distances, in feet, from Batter Board to Top of Pipe.

| | Date: 4/24/16 | | Date: 5/10/16 | Difference. | Distance, in feet, from point to point along center line of pipe. | Moved. | Location of punch mark. |
|---|---|---|---|---|---|---|---|
| *b*—1 | 0.855 | Supports removed after first observation. | 0.942 | 0.097 | ...... | 0.006 West | 0.82 East of bell. |
| *b*—2 | 0.818 | | 0.907 | 0.089 | ...... | 0.075 " | 0.77 " " " |
| *b*—3 | 0.781 | | 0.825 | 0.034 | ...... | 0.010 " | 0.75 " " " |
| *d*—1 | 0.873 | | 1.259 | 0.386 *b* to *d* | 13.71 | 0.009 West | 0.82 West of bell. |
| *d*—2 | 0.844 | | 2.350 | 1.506 *b* to *d* | 13.71 | 0.06 " | 0.95 " " " |
| *d*—3 | 0.790 | | 1.480 | 0.690 *b* to *d* | 13.73 | 0.015 " | 0.92 " " " |
| *e*—1 | 0.870 | | 1.360 | 0.490 *d* to *e* | 11.97 | 0.01 N. E. | 0.81 West of bell. |
| *e*—2 | 0.830 | | 2.988 | 2.158 *d* to *e* | 11.975 | 0.015 West | 0.85 " " " |
| *e*—3 | 0.812 | | 1.760 | 0.948 *d* to *e* | 11.98 | 0.010 East | 0.75 " " " |
| *f*—1 | 0.802 | | 1.160 | 0.358 *e* to *f* | 12.285 | ........... | 0.97 West of bell. |
| *f*—2 | 0.782 | | 2.217 | 1.435 *e* to *f* | 12.36 | ........... | 1.155 " " " |
| *f*—3 | 0.770 | | 1.401 | 0.631 *e* to *f* | 12.45 | 0.015 East | 1.175 " " " |
| *g*—1 | 0.847 | | 0.890 | 0.043 *f* to *g* | 12.72 | 0.001 West | 1.62 West of bell. |
| *g*—2 | 0.812 | | 0.843 | 0.031 *f* to *g* | 12.63 | ........... | 1.725 " " " |
| *g*—3 | 0.758 | | 0.792 | 0.034 *f* to *g* | 12.52 | ........... | 1.62 " " " |

Line No. 1 has developed absolutely no leaks, even under the additional load of 200 lb. per length. Lack of time has prevented loading and observing the effect of such loading to destruction of the pipe or joint. This will be done later.

Fig. 11 shows how the three mains were laid. The letters refer to the length of pipe under which they appear. The punch marks were

Mr. Randlett.

lettered according to the length of pipe on which they occur, and the joints were lettered according to the length of pipe containing the punch mark. The longitudinal distance was measured from punch mark to punch mark.

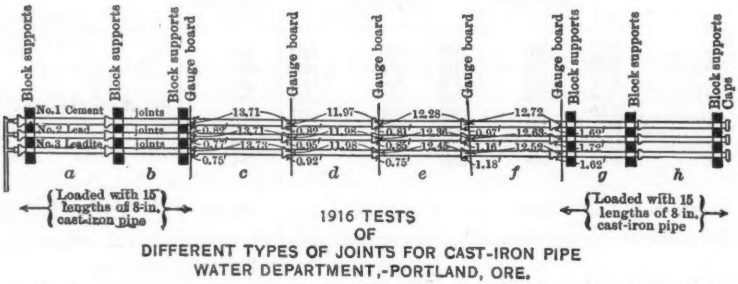


FIG. 11.

The cost per pound of the joint material was: Hemp, \$0.03; lead, \$0.09; leadite, \$0.12; cement, \$0.0055.

The cost per joint for material was about as follows:

| | | | | | | |
|---|---|---|---|---|---|---|
| *Line No. 1:* | Hemp, | 0.20 lb. | at | \$0.003 | \$0.006 | |
| | Cement, | 3.5 " | " | 0.0055 | 0.01925 | |
| | | | | | | \$0.02525 |
| *Line No. 2:* | Hemp, | 0.45 lb. | at | \$0.03 | \$0.0135 | |
| | Leadite, | 4.00 " | " | 0.12 | 0.48 | |
| | | | | | | 0.4935 |
| *Line No. 3:* | Hemp, | 0.45 lb. | at | \$0.03 | \$0.0135 | |
| | Lead, | 13.00 " | " | 0.09 | 1.1799 | |
| | | | | | | 1.1934 |

The average weight per foot of the 12-ft. lengths of pipe, including the bells, was:

*Line No. 1:* 567.5 lb. per length = 47.25 lb. per ft.
*Line No. 2:* 564.25 " " " = 47.0 " " "
*Line No. 3:* 573.625 " " " = 47.75 " " "

The actual weight of pipe suspended was:

*Line No. 1:* 2 288 + 800 (sand) = 3 088 lb.
*Line No. 2:*.......................2 326 "
*Line No. 3:*.......................2 311 "

In the case of Line No. 1, this would indicate, with the superimposed load of 200 lb. per length, an extreme fiber stress of about 9 000 lb. per sq. in. This may be within the elastic limit of cast iron, which varies from 6 000 to 20 000 lb. per sq. in.

Mr. Randlett.

*Conclusions.*—The conclusions, from these experiments are that for all ordinary mains, cement joints are superior to either lead or leadite; that leadite may often be used to advantage when time for setting of cement is not allowable; and that there are conditions where a lead joint might pull or blow out without breaking the pipe, and could be re-caulked and the main put in service more quickly than if cement were used.

The cost of making up joints is apparently in favor of leadite, with the cement joint next, and the lead joint the highest.

Mr. Shaw's statements in regard to the mixture of water and cement are substantiated by the work of Portland, except that the addition of from 15 to 20% of fine sand facilitates the ramming of the joint, and apparently does not materially weaken it. In dry weather it is necessary to keep the joint wet outside and inside, if possible, from 24 to 48 hours. If the joint is allowed to dry while setting, during the first few hours, shrinkage takes place, and this may or may not be taken up under pressure.

It is the custom of the Portland Water Department to test all lines to 50 lb. in excess of the normal pressure before putting the line in service, and it may be said, for the cement joints and workmanship, that only one of those made in 1916 leaked under test.

The City of Portland has about 6 miles of 8 and 12-in. cast-iron pipe laid with leadite joints since 1912, and no leaks have ever been reported. Many of the joints sweat slightly when the pressure is first turned on, but they take up rapidly and become absolutely tight in a few days.

Most of the objection to the use of cement and leadite comes from workmen who have long been accustomed to the use of lead, but the writer has not heard of any complaint from engineers or foremen who have given either a fair trial. The Portland Water Department contemplates a very general use of cement joints in future work.

It is hoped Mr. Shaw's paper will bring out additional facts and opinions on this subject.

Mr. Pearl.

WALTER PEARL,* M. AM. SOC. C. E. (by letter).†—In these days of Twentieth Century progress, and under the present duress of military requirements concerning our minerals and metals, this paper presents much matter for interesting discussion, in the way of economy, along the line of public work, as the latter must be continuous, and extend indefinitely, as long as civilization exists, regardless of whether the world is at peace or only striving for peace.

During several years of hydraulic engineering practice, embracing water-works construction for municipalities, the writer has noted the

* Los Angeles, Cal.

† Received by the Secretary, April 17th, 1917.

Mr. Pearl. rapid exchange or substitution of the baser materials, metals, or elements, for those of refined, or combination metals and construction materials; in the case of wood, timber, and lumber, more metal is being used in the constructive combination.

Recently, bids were received in the State of Washington for a steel highway bridge, but as it was afterward found that a concrete bridge would cost only a trifle more, a bid was accepted for work of this class, considering the small margin in favor of the permanence of a concrete structure. There have been many similar instances lately, and as all branches of the Government and all individuals must now practice economy, this paper offers suggestions along many lines.

The method of making the cement joints, is described very clearly and concisely, including the method of breaking or loosening them, all of which would necessitate less time and expense, apparently, than is ordinarily required in melting, pouring, and driving lead joints, after caulking the joint with oakum or hemp in the usual manner, or the necessary expense and labor of melting the lead from the joints in disconnecting them; these elements of time and labor are favorable to the cement joint. However, more data and experience with the cement joint should be available before final conclusions can be reached regarding the relative cost of the two kinds of joints; former data cannot be used at the present time in making up a table showing the cost of lead joints.

It is a question with the writer whether the cement, as described, is sufficiently wet to fill all the joint space, and, certainly, great care must be taken in driving or caulking the cement, as the joint space is limited, and irregularities in the pipe often provide very small space, even for hot lead.

Table 5, showing the joint space, etc., in cast-iron pipes, is taken from the catalogue of a manufacturer of cast-iron pipes.

TABLE 5.—Some Dimensions of Cast-Iron Water Pipe.

(Thickness of shell proportioned for 100 lb. static pressure.)

| Diameter, in inches. | Length, over all. | Thickness of shell, in inches. | Depth of hub, in inches. | Joint room, in inches. |
|---|---|---|---|---|
| 4 | 12 ft. 4 in. | 7/16 | 3 | 5/16 |
| 6 | 12 " 4 " | 1/2 | 3 | 5/16 |
| 8 | 12 " 4 " | 17/32 | 3 | 5/16 |
| 10 | 12 " 6 " | 19/32 | 3 | 5/16 |
| 12 | 12 " 6 " | 5/8 | 3¼ | 5/16 |
| 14 | 12 " 5 " | 11/16 | 3¼ | 5/16 |
| 16 | 12 " 5 " | 3/4 | 3½ | 3/8 |
| 18 | 12 " 5 " | 25/32 | 3½ | 3/8 |
| 20 | 12 " 5 " | 27/32 | 3½ | 3/8 |
| 24 | 12 " 5 " | 15/16 | 3¾ | 3/8 |

The figures in Table 5 may be assumed as standard, but as the flask, mould, and pattern vary slightly with different manufacturers, the "joint room" may suffer. Table 5 shows what a limited space there is in the hub or bell for cement after the spigot end is entered, for a thickness of not less than $\frac{1}{4}$ to $\frac{1}{2}$ in. of cement should be required throughout the entire joint. This thickness might prevent the possibilities of seepage or leaks in the joints of the pipe constructed under the author's supervision. The paper gives no information regarding the joint space. In laying a line of pipe such as described, great care would be necessary in securing, as nearly as possible, perfect alignment and grade, in order to have uniform joint space. Mr. Pearl.

It would seem that cement joints could only be used with straight lines of pipe under low pressures. In a joint of a 6-in. cast-iron pipe, when laid straight, the space is $\frac{5}{16}$ in., and on a curve of 250 ft. radius, the resulting space would be practically $\frac{3}{16}$ in., or $\frac{1}{8}$ in. on a curve of 166 ft. radius. Numerous ups and downs in grade could not be permitted, thus showing the necessity of laying such pipe according to the method of laying sewer pipe. This matter of joint space has been gone into at some length by the writer, as he realizes the difficulty of pouring the lead in a close joint, and more especially in caulking it.

Another question arises, as to whether cement joints in a cast-iron main will stand the strain and shocks due to the sudden closing of valves and hydrants (though not necessary, such things sometimes happen), causing water-hammer, ram, and vibration in the line of pipe, which might crack or injure the cement joints, causing leakage, expense, and annoyance. This is likely to occur in a gravity pipe line, and, in a pumping line, the constant vibration might cause seepage and leaks to develop in the cement in due time. It is well known that cement and concrete will crack and disintegrate where there are sudden shocks or continuous vibration, though it will stand great pressures due to head or weight, if not subject to disturbance.

The author has described cases where pipes with cement joints were laid in made or filled ground, and where settlement occurred without injuring the joints or causing leakage; also where a parallel trench caused the caving of the pipe trench, leaving the pipe hanging unsupported in the air for a time, without even causing seepage in any of the joints. The reason for this, doubtless, was the gradual giving way of the ground supporting the main. Had the shock been sudden, the results might have been different.

It may be conceded that no other base metal having the qualities of lead—so ductile and homogeneous—seems to be as well adapted for the joints of cast-iron pipe under pressure. Lead joints may be driven or re-caulked, and all leakage stopped, while the pressure is on, which is quite an advantage over cement, as it would be impos-

Mr. Pearl. sible, practically, to secure any bond with the cement after it had thoroughly set. In case of an important leak it would be necessary to shut off all pressure on the main, and possibly the water would have to be drawn off; then, after removing all the original cement, an entire new joint would have to be made.

In a grade-line pipe, there is no doubt that cement joints would be safe and economical; but in a high-pressure cast-iron main, it would appear to the writer to be difficult to find a substitute for lead joints; under the present, prevailing high prices of materials, and especially metals, the cost and the special requirements relative to the safety of the structure, should be well considered before work begins.

It is possible that this paper may bring out some discussion relative to the likelihood of electrolysis along a cast-iron main with cement joints paralleling electric car lines; it would seem that, as cement is a non-conductor, such a line of pipe would be unfavorable to electrolytic action, as the current being so frequently broken would leave the main in minor quantities without causing deterioration; however, experiments would demonstrate this matter more satisfactorily.

The author states that the cement joint for cast-iron pipe has passed the experimental stage, especially in work with which he has been connected, and that such joints are safe and satisfactory. The work seems novel to the writer, and he is indebted to the author for attracting his attention to new construction methods.

Mr. Moulton. H. G. MOULTON,* M. AM. SOC. C. E.—With respect to the use of Portland cement instead of lead in forming joints for cast-iron water mains, it may be said that this is the method against which there is every theoretical objection, but in favor of which there is the practical argument that it has become a demonstrated success in actual practice. For a number of years past the use of cement for this purpose has been standard practice in Los Angeles, a thriving municipality of some 350 000 population, and the speaker is indebted to William Mulholland, M. Am. Soc. C. E., Chief Engineer of the Bureau of Water Works and Supply of that city, for much interesting information in regard to this method of making joints. The following statements in regard to costs and conditions in Los Angeles are in a large measure based on figures furnished by Mr. Mulholland.

The advantage in favor of this method lies in its great economy. In 1912 about 9 000 ft. of 30-in. high-pressure water main were laid in Los Angeles. Cement at that time was quoted at $2.00 per bbl. there and lead cost 5 cents per lb. The total saving on the job by using cement instead of lead was approximately $3 500. At present comparative prices of lead, cement, and labor, the saving would be

* New York City.

very much greater, and, of course, on 36 or 48-in. mains a very large saving is possible. Mr. Moulton.

An additional advantage in the use of cement lies in its insulating effect, in that it appears to act as a perfect seal between the separate sections of pipe and thus to reduce materially the effect of electrolysis. As damage to cast-iron pipe from stray electric currents has resulted in deterioration of water mains in many places, any type of joint which tends to reduce such damage, by stopping the flow of stray current along the pipe, is worthy of serious consideration.

The arguments against the use of the cement joint are based on a fear that temperature changes, resulting in uneven expansion and contraction, would tend to break up the joint, and also on doubt as to the action of pipe caulked by this method in the event of settlement, in filled ground or otherwise. In Los Angeles, the usual practice is to refrain from the use of cement joints where pipe must be laid on fills, probably on the assumption that, in case of settlement opening up joints, they can be re-caulked more easily if lead is used. In Long Beach, however, as mentioned in the paper, the cement joint has been used in filled ground, and also under conditions where trenches adjacent to the pipe have allowed sections of it to sag over a length of 40 ft. in one instance; and another instance is mentioned in the paper where some 98 ft. of pipe broke away and dropped into a trench with all the joints remaining in perfect condition, except those at the actual point of rupture.

In regard to the question of the ability of pipe to span extensive distances when the supporting ground is washed away, it may be said that, in designing connections for water pipe lines, good practice does not call for laying them out as suspension bridge connections, under the assumption that the pipe should be able to hang suspended over extensive spaces without failure. Under all normal conditions, a water pipe joint need only be considered in regard to its ability to prevent the leakage of water with reasonably continuous support and with a proper depth of cover above. Its behavior under abnormal conditions, where it has to hang suspended over wash-outs or cave-ins, is a matter of interest only from a standpoint of curiosity, and one is certainly not justified in designing all pipe on the assumption that it must meet conditions such as this, and providing special connections for this purpose.

In regard to the question of temperature changes, it may be said that there is seldom a variation of more than 50° between the winter and summer temperatures in city water mains. In winter, temperatures lower than 32° Fahr. cannot exist, on account of the fact that, at this point, the water changes to ice; and above a temperature of 85°, it is certainly too hot for use as drinking water. In Los Angeles the maximum variation in temperature is from 45° Fahr. in winter to 82°

Mr. Moulton.

in summer, or a total range of 37° Fahr. Under this range, no trouble has been experienced with cement joints from leakage introduced by temperature stresses.

This paper brings up an interesting method which was first devised in California, and the success of which has been proved there on an extensive scale. It is no longer in an experimental stage, and is worthy of serious consideration on the part of eastern municipalities. The saving in expense made possible by the substitution of cement for lead in the joints of cast-iron pipe is so great that all the larger eastern cities would be justified in commencing immediately the use of cement joints in an experimental way in outlying districts, working gradually in to more important parts of the water system as the advantages and limitations of the method are developed in each municipality.

The author is entitled to a large measure of credit for having brought thus forcibly before the Society the advantages of a pioneer engineering method having great possibilities, which the Engineering Profession as a whole has been slow to recognize and adopt.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By Messrs. H. V. Hinckley and Henry T. Eddy.

Mr. Hinckley.

H. V. Hinckley,† M. Am. Soc. C. E. (by letter).‡—Three points pertaining to "coarse aggregate" occur to the writer as being possibly worthy of mention, namely, large aggregate, glassy aggregate, and strong aggregate.

That "a large aggregate produces a stronger concrete than a fine one" will not be questioned by any competent engineer. It is fortunate, however, that the point has been brought out, because public officials are apt to look on coarse aggregate as a shirking of duty on the part of the rock crusher rather than the exercise of good judgment by the engineer.

The following substitute clause is offered as a suggestion: "The Committee does not feel warranted in recommending the use of blast-furnace slag," flint, or other aggregates which produce smooth, glassy faces, especially for reinforced concrete in which high strains are to be developed.

It is fully as important that the coarse aggregate shall be of tough, strong rock, as it is that "fine aggregate should always be tested for strength." Of two classes of rock (other conditions being equal), the class that, as a beam, will support the heavier load will make, if crushed for coarse aggregate, the stronger concrete. When concrete fails under compression, it fails in tension, and, in failing, it generally

* Continued from April, 1917, *Proceedings*.

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‡ Received by the Secretary, April 10th, 1917.

Mr. Hinckley. splits through the coarse aggregate rather than around it, and as the rock thus broken makes up the greater portion of the section of failure, the strength of the concrete is largely dependent on the strength of the stone. The writer recently stated*:

"Every engineer of experience should know that, under any ordinary concrete specifications, he can build two slabs side by side, using the same sand, the same cement, the same proportion, the same time of mix in the same machine, and at the end of 90 days have one slab at least twice as strong as the other."

If the writer had to build a reinforced concrete structure, in which high strains were to be developed, he would pay as much attention to the modulus of rupture of the rock to be used as he would to the tests of the fine aggregate. It is time for the Society to get away from the idea that any old stone will make good concrete. A "clean, hard, and durable" stone may be brittle and weak, and it might not be out of place to add, at least, the word "strong."

Since writing the foregoing, the writer's attention has been called to Professor George A. Hool's "Reinforced Concrete Construction", in which is found the only corroboration of the foregoing views which he has ever noticed. Professor Hool says: "Any stone is suitable which is clean and durable, and has sufficient strength to prevent the strength of the concrete from being limited by the strength of the stone," and this means that the stone must be as strong as the mortar.

Mr. Eddy. HENRY T. EDDY,† ESQ. (by letter).‡—*1.—The Thickness of Flat Slabs.*—In the first place the writer desires to consider somewhat critically the formula recommended in the report for the value of the minimum total thickness, $t$, in inches, of a flat slab without dropped panels, in order to show that it is a formula not suited to give the minimum thickness according to accepted principles in reinforced concrete design. The formula recommended in the report is:

$$t = 0.024\, l \sqrt{w} + 1.5 \qquad (1)$$

in which $l$ = panel length or span between column centers, in feet, and $w = w_0 + w_1$, the sum of the live load, $w_0$, and the dead load, $w_1 = 12.5\, t$ lb. per sq. ft. Similar formulas have been proposed in the Chicago Ruling§ and in the standard building regulations reported to the American Concrete Institute.‖

Equation (1) consists of two parts, the mean effective moment thickness, $d$, or the depth of the center of the steel below the compressed

* *Transactions*, Oklahoma Soc. of Engrs., Vol. III.

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‡ Received by the Secretary, April 23d, 1917.

§ *Engineering News*, September 24th, 1914.

‖ *Proceedings*, Am. Concrete Institute, Vol. XIII, 1917.

slab surface, and 1.5 in., which last includes half the thickness of the steel mat of crossed rods plus concrete fire-proofing at least 1 in. thick. Hence, we have the effective moment thickness Mr. Eddy.

$$d = 0.024\ l\ \sqrt{w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

an equation which, together with Equation (1), the writer has reason to believe is so far from being correct that it should not be recommended or regarded as expressing the minimum value, a conclusion that follows from considerations that will now be given in detail.

The general expression for the bending moment, due to the applied forces acting at any panel-wide vertical section of a square panel of span $l$ ft., when loaded with a total uniformly distributed load of $w$ lb. per sq. ft., or a total load of $W = wl^2$ lb. per panel, may be written in the form:

$$12\ W \frac{l}{a} = 12\ w \frac{l^3}{a} \text{ in-lb} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which $a$ is a numerical divisor the value of which has frequently been recommended to be taken at about 24 or 25 for the section at mid-span and at $-12$ or $-15$ at the edge of the panel, with intermediate values, positive or negative, at sections situated between these. These, however, are the two sections which are most frequently taken into consideration.

Again, the general expression for the resisting moment of the steel that crosses any vertical cross-section of the panel which has an effective thickness of $d$ in. and a total effective cross-section of steel of $A = 12\ pld$ sq. in., may be written in the form:

$$f_s A j d = 12 f_s p l j d^2 \text{ in-lb} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which $f_s$, in pounds, is the unit stress in the steel, $jd$ is the moment arm, $12l$ is the panel length, in inches, and $p$ is the steel ratio of the section.

It is assumed ordinarily that when the direct tensile resistance of the concrete acting in parallel with the tensile resistance of the steel is neglected, the resisting moment must be equal to the applied moment. The writer, however, has reasons which he has explained elsewhere for believing that the applied moments are more than twice as large as the actual resisting moments shown by observation. In case this is the fact, it would only be necessary, in order to make Equations (3) and (4) equal, to assume as the numerically correct value of $a$, a number which is the same multiple of the value of $a$, ordinarily assumed, that the applied moments are of the resisting moments. However, leaving this question unsettled for the present, and leaving also the value of $a$, which will be finally adopted as applicable at mid-span, undetermined, we may nevertheless equate the resisting moment to the applied moment, with the understanding that such

Mr. Eddy. a value of $a$ will finally be adopted as may appear to be required by the facts. Hence

$$f_s p j d^2 = w \frac{l^2}{a},$$

or,

$$d \doteq \frac{l\,w}{\sqrt{f_s j p a}} \qquad \text{.......................(5)}$$

Now compare Equation (5) for the effective thickness, $d$, with Equation (2), which was recommended in the report. In case Equation (2) is correct, it must be identical with Equation (5). To express this relation more concretely, assume that $f_s = 16\,000$ lb. per sq. in., and that $j = 0.9$ at the mid-section. Then, in order that Equations (5) and (2) may be identical, we have

$$16\,000 \times 0.9 \times (0.024)^2 pa = 1, \text{ or } pa = 0.12 \qquad \text{........(6)}$$

Hence, when we

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| take | $a =$ 12, | 15, | 20, | 24, | 30, | 40, | 50, | 60, |
| we then have | $p =$ 0.01, | 0.008, | 0.006, | 0.005, | 0.004, | 0.003, | 0.0024, | 0.002, |

respectively.

It thus appears that in case Equation (2) is, in fact, a correct form of expression for $d$, then Equation (6) must also hold true; but as $a$ in Equation (6) is a constant which is fixed by the position where the section is taken, at mid-span or elsewhere, and, moreover, is independent of both $l$ and $w$, it follows from Equation (6) that $p$ is also constant so long as $a$ remains constant. If there is anything in the design of concrete beams, however, that has been indubitably established by experience, it is that the steel ratio increases with the relative thickness, that is, $p$ increases with $\frac{d}{l}$ and the same principle applies to slabs also. Hence, Equations (1) and (2) are not, in fact, correct, and cannot be relied on to fix the minimum value of $d$, because they require a constant value of $p$, not in accordance with experience. In fact, $p$ is found to be nearly twice as large in deep slabs for heavy loads as it is in shallow slabs for light loads, in case of well-designed slabs for any given span.

The reason for such increase in the value of $p$ in case of deep beams is evident from the following considerations. In shallow beams and slabs the sharpness of the curvature is inversely proportional to $d$ for a given limiting compression in the concrete at the top surface. The sharper the curvatures the greater the horizontal shearing distortions in the concrete at points distant from the sections where maximum moments occur. Concrete affords a resistance to shearing distortion which is small compared with its resistance to compression. In shallow beams high values of $p$ are futile, for this reason, but, in

deep beams, larger values of $p$ may be used than in shallow beams, on account of their smaller shearing distortion. In slabs this still further reason may exist for an increase of $p$ with $\frac{d}{l}$, that is, the crossed rods that make up the mat may properly be more numerous and more closely spaced in a thick slab than in a thin slab, thus furnishing more effective co-action of steel and concrete in thick than in thin slabs. Mr. Eddy.

In its present form Equation (2) is not convenient for numerical calculations, because the value of $w$ itself depends on $d$. It will be more convenient to express $d$ as a function of $w_0$, the live load. Write Equation (2) in the form,

$$d = 0.024\, l \sqrt{w_0 + w_1}$$

by the help of the equation

$$w = w_0 + w_1 \text{.......................} (7)$$

and eliminate

$$w_1 = 12.5t = 12.5\ (d + 1.5)$$

by assuming the weight of a cubic foot of concrete to be 150 lb., or 12.5 lb. per in. of thickness per square foot.

Then,

$$d^2 = (0.024\, l)^2\ (w_0 + 12.5\, d + 18.75).$$

Solve this quadratic equation and find,

$$d = 0.00375\, l^2 + 0.024\, l \sqrt{(w_0 + 18.75 + 0.00225\, l^2)} \text{.....} (8)$$

Table 1 has been computed from Equation (8). Now, if the value of the divisor in Equation (3) is assumed to be $a = 25$ at mid-span of the panel, as recommended in the report, then, by Equation (6), we find the steel ratio, $p = 0.0048$, throughout the entire table, regardless of $\frac{d}{l}$. It is apparent that the thickness found in Table 1 is too great at the larger spans and loads, so much so as almost to prohibit the use of flat slabs for the larger loads and spans figured in the table, but it is evident that this thickness may be reduced by increasing $p$ somewhat with $\frac{d}{l}$.

The effect of this increase of $p$ with $\frac{d}{l}$ will evidently be to produce a proportionate decrease of the dead load and of the thickness of the slab at the larger values of $w_0$ over that at the smaller values of $w_0$.

With the view of finding a simple approximate expression for $d$ that will give values of $d$ more nearly in accordance with the results of good practice than those obtained from Equation (4) or from Table 1, it is to be observed that, for any given span, $l$, Equation (8), re-

Mr. Eddy. garded as a quadratic expressing the relation between the variables, $d$ and $w_0$, taken as co-ordinates, represents a parabola. The part of the curve in question ranges from $w_0 = 100$ or less, to $w_0 = 600$ or more. In Fig. 7 the values of $d$ in this range are plotted for $l = 20$ ft. Similar curves occur for other values of $l$.

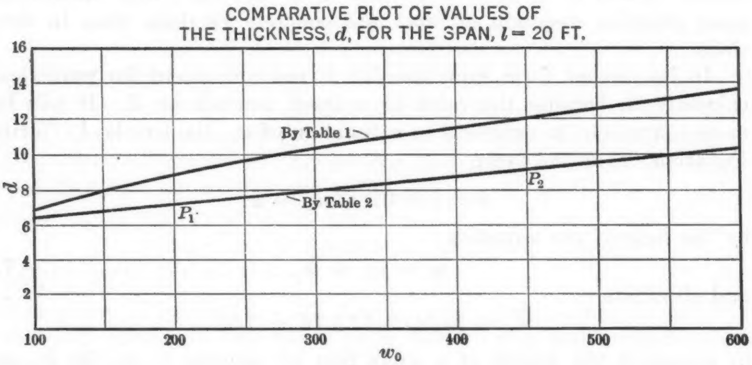


FIG. 7.

Suppose that the effective thickness for two different loads as, for example, at $P_1$ and $P_2$ in Fig. 7, were sufficiently well ascertained by a consensus of good practice to be supposed to be known, then it would be possible to obtain an equation for a straight line through those two points that would give values of $d$ for those and other loads, which

TABLE 1.—EFFECTIVE THICKNESS OF FLAT SLABS.

$$d = 0.00375\ l^2 + 0.024\ l \sqrt{(w_0 + 18.75 + 0.0225\ l^2)} \text{ inches;}$$

$l$ = span, in feet; $w_0$ = live load, in pounds per square foot.

| Span, in feet. | $w_0 = 100$ | 200 | 300 | 400 | 500 | 600 |
|---|---|---|---|---|---|---|
| 14 | 4.46 | 5.75 | 6.78 | 7.64 | 8.42 | 9.13 |
| 16 | 5.24 | 6.72 | 7.87 | 8.87 | 9.75 | 10.56 |
| 18 | 6.06 | 7.71 | 9.02 | 10.13 | 11.13 | 12.03 |
| 20 | 6.92 | 8.74 | 10.19 | 11.42 | 12.52 | 13.53 |
| 22 | 7.82 | 9.81 | 11.40 | 12.74 | 13.96 | 15.06 |
| 24 | 8.76 | 10.92 | 12.65 | 14.12 | 15.44 | 16.62 |
| 26 | 9.75 | 12.08 | 13.93 | 15.54 | 16.95 | 18.25 |
| 28 | 10.78 | 13.27 | 15.26 | 16.99 | 18.50 | 19.89 |
| 30 | 11.86 | 14.49 | 16.02 | 18.45 | 20.07 | 21.57 |
| 32 | 12.98 | 15.77 | 18.04 | 19.98 | 21.71 | 23.29 |

might be more nearly in accord with good practice than those given by the parabola. Such an equation of relation between the effective thickness and the live load has been derived in what follows.

The equation for $d$ through $P_1$ and $P_2$ may then be written in the form, Mr. Eddy.

$$d = l \frac{w_0 + b}{c} \qquad (9)$$

in which $l$ is the span, from center to center of supports, in feet, and $w_0$ is the total design working load, in pounds per square foot. The total thickness is

$$t = d + 1.5 \qquad (10)$$

in which, if $d$ is the effective depth of the centers of gravity of the steel below the compression surface of the concrete, 1.5 in. is the remaining part of the total thickness, including one-half of the layer of steel and its fire-proof covering. In the report this is taken as 1.5 in., of which the fire-proofing is required to be 1 in. in the clear.

In Fig. 7 the two points, $P_1$ and $P_2$, at which it was assumed that the two corresponding values of $d$ were known, were both for the same span, $l$, but the values of the constants, $b$ and $c$ in Equation (9), can be determined equally well in case the known values of $d$ correspond to any different known values of $l$ and $w_0$ whatever. For example, assume that it is known from experience that, for a 17-ft. panel and a design live load of 200 lb. per sq. ft., the total thickness, $t$ = 7.6 in., nearly, and for a 23-ft. panel and a design load of 400 lb. per sq. ft., $t$ = 11.6 in., nearly. Substituting these values in Equation (9), gives the following equations from which to determine the values of $b$ and $c$:

$$\left.\begin{aligned} 7.6 - 1.5 &= 17 \frac{200 + b}{c} \\ 11.6 - 1.5 &= 23 \frac{400 + b}{c} \end{aligned}\right\} \qquad (11)$$

Hence, $b$ = 700 and $c$ = 2 500, nearly, and Equation (9) becomes

$$\left.\begin{aligned} d &= l \frac{w_0 + 700}{2\,500} \\ \text{or,} \quad d &= l \frac{4\,w_0 + 2\,800}{10^4} \end{aligned}\right\} \qquad (12)$$

the latter form being more convenient for numerical computation.

In case the examples previously assumed as a basis for the numerical values of $b$ and $c$ used in Equation (12) seem to be unacceptable, then any two other trustworthy and satisfactory numerical examples may be used instead, to arrive at an equation with values of $b$ and $c$ slightly different from those used in Equation (12). Table 2, giving the numerical values of $d$, has been computed from Equation (12).

The differences in the successive numbers in any row or column of Table 2 are constant, and are noted in the margins at the right and

Mr. Eddy. bottom of the table, so as to admit of ready interpolation and exterpolation by proportional parts, which, in this case, is exact and not merely approximate. If, in Equation (12), $w_0$ and $l$ are regarded as rectangular co-ordinates in a horizontal plane, and $d$ is erected perpendicular to it, the locus of the extremity of $d$ will be a hyperbolic paraboloid which by its height will represent graphically the thickness of the slab. The numbers at the top and left of Table 2 measure such co-ordinates in the plane of the paper, and the numbers in the body of the table express the heights to be laid off at the positions where the numbers are written. The graphical surface thus constructed is a doubly-ruled surface such that a vertical section of it, for any given constant value of either $w_0$ or $l$, will be a uniformly sloping straight line.

The total thickness of the slab will be obtained as in Equation (10) by adding to $d$ one-half the total thickness of the layers of reinforcing steel plus the thickness of the fire-proofing, which all together may be either more or less than the average value of 1.5 in. recommended in the report.

The values of $d$ in Table 2 are such as to make the thickness, $t$, throughout the entire table fall within the limit of one thirty-

TABLE 2.—EFFECTIVE THICKNESS OF FLAT SLABS.

$$d = l\,\frac{4\,w_0 + 2\,800}{10^4}\ \text{inches;}$$

$l$ = span, in feet; $w_0$ = live load, in pounds per square foot.

| Span, in feet. | $w_0 = 100$ | 200 | 300 | 400 | 500 | 600 | Δ |
|---|---|---|---|---|---|---|---|
| 14 | 4.48 | 5.04 | 5.60 | 6.16 | 6.72 | 7.28 | 0.56 |
| 16 | 5.12 | 5.76 | 6.40 | 7.04 | 7.68 | 8.32 | 0.64 |
| 18 | 5.76 | 6.48 | 7.20 | 7.92 | 8.64 | 9.36 | 0.72 |
| 20 | 6.40 | 7.20 | 8.00 | 8.80 | 9.60 | 10.40 | 0.80 |
| 22 | 7.04 | 7.92 | 8.80 | 9.68 | 10.56 | 11.44 | 0.88 |
| 24 | 7.68 | 8.64 | 9.60 | 10.56 | 11.52 | 12.48 | 0.96 |
| 26 | 8.32 | 9.36 | 10.40 | 11.44 | 12.48 | 13.52 | 1.04 |
| 28 | 8.96 | 10.08 | 11.20 | 12.32 | 13.44 | 14.56 | 1.12 |
| 30 | 9.60 | 10.80 | 12.00 | 13.20 | 14.40 | 15.60 | 1.20 |
| 32 | 10.24 | 11.52 | 12.80 | 14.08 | 15.36 | 16.64 | 1.28 |
| Δ = | 0.64 | 0.72 | 0.80 | 0.88 | 0.96 | 1.04 | |

second of the span, as recommended in the report, excepting only the last number at the bottom of the first column, that is, $t = 10.24 + 1.5 = 11.74$, which is $\frac{1}{4}$ in. below the assigned limit.

It will be noticed that Tables 1 and 2 give practically identical values of $d = 4.5$ in., or $t = 4.5 + 1.5 = 6$ in., at the upper left

corner, where $l = 14$ ft., $w_0 = 100$ lb.; and at the upper right and left corners the values of $d$ in Table 1 exceed those in Table 2 by about 25%, and at the lower right corner by nearly 50 per cent. These enormous differences are due to the increase in the value of $p$ with $\frac{d}{l}$ involved in Equation (5), when $a$ is constant. Mr. Eddy.

When the value of $a = 25$ is used, that necessarily implies that the slab has steel enough in one direction to carry the entire load without any stress in the steel at right angles thereto, whereas, in fact, in an inside panel, the steel in both directions acts simultaneously, which requires $a = 50$ and not $a = 25$. This fact will reduce the value of $p$ to one-half that previously found.

The steel ratio derived from Equation (5) is:

$$p = \frac{w\,l^2}{a\,f_s\,j\,d^2} = \frac{(w_0 + 12.5\ d)\ l^2}{16\ 000 \times 0.9\ a\ d^2} \qquad (13)$$

which gives the range of values of $p$, shown in Table 3. This table gives the numerical value of the ratio of the cross-section of the steel to the concrete at mid-span, in order that, when $a = 25$, the steel will resist the entire applied moment as in a beam, or, when $a = 50$, the steel will resist one-half of that amount, as in a two-way slab. Any intermediate values of $l$ and $w_0$ may be found correctly from this table by simple interpolation of proportional parts.

TABLE 3.—VALUES OF STEEL RATIO, $p$, BY EQUATION (13).

| | IF $a = 25$ | | IF $a = 50$ | |
|---|---|---|---|---|
| | $w_0 = 100$ | $w_0 = 600$ | $w_0 = 100$ | $w_0 = 600$ |
| $l = 14$ | 0.0047 | 0.0073 | 0.0024 | 0.0036 |
| $l = 32$ | 0.0067 | 0.0083 | 0.0034 | 0.0042 |

These values of $p$ show that Table 2 requires no excessive steel ratios, either at mid-span or at panel margins, where, since the values of $a$ will be about one-half those at mid-span, the values of $p$ will be about twice as great.

In obtaining the values of $d$ in Table 2, no allowance has been made for any reduction of span by reason of size of capitals. In order to make such allowance, it will be sufficient to assume that $l$ is the effective span, instead of the actual span, and use it as the tabular span for finding $d$ in Table 2.

It will now be in order to find out, in case slab thicknesses in accordance with Equation (9) and Table 2 are adopted, whether either the

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compressions in the concrete will be excessive, or the deflections of the slab will be greater than is permissible. For, if neither of these things occurs, then we have established the admissibility of these thicknesses, which are less than those found by Equation (1) and Table 1, and, hence, these latter will not be actual minimum values, but should be replaced by other smaller values found from Equation (9). The compression in the concrete is to be found from the common formula, $f_c = \frac{f_s\, k}{n\,(1 - k)}$, in which $k$ must be less than 0.4, and probably does not exceed 0.3; and we may assume $f_s = 16\,000$, and $n = 15$, so that the unit stress, $f_s$, does not exceed 700 lb., and probably is not more than 500 lb., at any point, unless at the edge of the capital, at which point a special investigation may be necessary. This shows that, in general, the compression in the concrete should not be regarded as excessive.

An expression for the deflection, $z$, at the panel center of a flat slab has been found to be:

$$z = \frac{W\,L^3}{4.75 \times 10^{10}\, d^2\, A_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)^*$$

in which $L = 12\,l$ is the span, in inches (not feet), and $A_1$ is the cross-section of a side belt.

Now,

$$A_1 = \frac{1}{2}\,A = \frac{W\,L}{2\,f_s\, a j\, d},$$

which last was obtained by equating applied and resisting moments as in Equations (3) and (4). Substitute this value of $A_1$ in Equation (13), and assume $f_s = 16\,000$, $a = 25$, $j = 0.9$, and let the slab be as thin as permissible, namely, $\frac{L}{d} = 32$. We then have the largest possible relative deflection for a unit steel stress of 16 000 lb. Then, the relative deflection,

$$\frac{z}{L} = \frac{2\,f_s\, j\, a\, L}{4.75 \times 10^{10}\, d} = \frac{1}{2\,060} \quad \ldots\ldots\ldots\ldots\ldots (15)$$

This relative deflection is so small that it would permit a test load of more than 2.5 times the sum of the design load plus the dead load, without causing $\frac{z}{L}$ to exceed $\frac{1}{800}$, provided the steel were not thereby stressed beyond its elastic limit, which, in this case, would have to be at about 40 000 lb. per sq. in. Smaller values of $\frac{L}{d}$ than 32, such as

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* See "Concrete Steel Construction", Eddy and Turner, (71), p. 204.

occur throughout almost the entire Table 2, will give smaller values of $\frac{z}{L}$ than the limiting value computed in Equation (15). On the other hand, although a value of $a$ larger than 25 would have an effect to offset the smaller values of $\frac{L}{d}$ and so to increase $\frac{z}{L}$, yet the observed values of $f_s$ are invariably much smaller than those computed by Equation (14), which serves actually to reduce the value of $\frac{z}{L}$ at least to that found by Equation (15). It follows, consequently, that values of $d$ as small as those in Table 2 may be used with safety, and that values as large as those in Table 1 as prescribed by Equations (1) and (2), are not a minimum, and it has been further shown that they are unnecessarily large. Mr. Eddy.

The report recommends minimum total thicknesses that may be written as follows:

$$\left.\begin{array}{ll} \text{For a slab without a drop,} & t = 0.024\, l \sqrt{w} + 1.5 = d + 1.5 \\ \text{For slab with drop,} & t = 0.02\, l \sqrt{w} + 1 = \frac{5}{6} d + 1 \\ \text{For the drop itself,} & t = 0.3\, l \sqrt{w} + 1.5 = \frac{5}{4} d + 1.5 \end{array}\right\} \ldots (16)$$

and, further, that the width of the drop is 0.4 $L$, so that its area will be $(0.4\, L)^2 = \frac{L^2}{6}$, nearly. Hence, the total volume, in cubic inches, of a panel without a drop would be:

$$v = (d + 1.5)\, L^2 \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

but the volume of a panel together with a drop would be:

$$v = \left(\frac{5}{6} d + 1\right) \frac{5}{6} L^2 + \left(\frac{5}{4} d + 1.5\right) \frac{L^2}{6}, \text{ nearly, or}$$

$$v = \left(\frac{65}{72} d + 1.1\right) L^2 \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

From this it is evident that the report discriminates against the flat slab without a drop and in favor of the slab with a drop by making the latter so much thinner that the total volume per panel is less. In the writer's opinion such discrimination is without justification in theory or practice. In his opinion, in order to make the formulas for the thicknesses of slabs with a drop comparable with those without a drop, the volume of the former should be at least as great as the latter. With this in view, it will be necessary, in order to have the

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drop 50% thicker than the rest of the slab and its area $= \frac{L^2}{6}$ to write for the thickness of a slab with a drop,

$$t = 0.022\, l \sqrt{w} + 1.5 = \frac{11}{12} d + 1.5$$

and for the drop itself,

$$t = 0.0033\, l \sqrt{w} + 1.5 = \frac{11}{8}\, d + 1.5 \qquad \ldots\ldots\ldots\ldots (19)$$

Hence,

$$v = \left(\frac{11}{12} d + 1.5\right) \frac{5}{6}\, L^2 + \left(\frac{11}{8} d + 1.5\right) \frac{1}{6}\, L^2$$

or, $v = (d + 1.5)\, L^2$, nearly.

The reason for the foregoing statement is this: In the case of a given uniform loading, the numerical sum of the bending moments at mid-span and one margin is constant. but the sub-division of this constant total moment between mid-span and margin depends on the structure of the slab, as follows: Either the line of inflection in the slab occupies a position fixed by the simultaneous dip of the reinforcing rods as they drop below the neutral surface, or it is undetermined, due to the irregular dipping of the steel, and, in that case, is determined only by the relative stiffness of the columns and of the panel at the margin, as compared with that at mid-span. The report does not contemplate any fixed lines of inflection, for it makes express provision for irregular dipping of steel rods. However, it recommends that steel be provided to resist moments of certain given magnitudes at mid-span and margin separately, regardless of the presence or absence of a drop. If that recommendation is to be adopted, the suspended portion of the panel must needs be just as thick in case of a drop as without it. The only excuse for its being made thinner with a drop would be that the columns and the cantilever portion containing the drop had been made stiffer and the cantilever made larger thereby, so that the lines of inflection were thereby made to occupy positions somewhat more distant from the columns than in the case where there is no drop. This would require the steel over the drop to be kept near the top of the slab for a greater distance than in a plain flat slab, and would also require larger and stiffer columns to resist the unequalized moments, but would hardly permit the total volume of the panel to be decreased, as has been done in the report.

It is still an unsolved problem to determine the economical relative sizes of the suspended portions of the panel and of the cantilever, and the relative stiffness of the columns, as well as the manner of distribution of the resisting moments across the margins of the panels. On the solution of this problem evidently no light has been shed by the recommendations of the report, since it recommends a difference

in thickness without any difference in the resisting moment—a recommendation hard, if not impossible, to justify. Mr. Eddy.

Equations (19) for thicknesses of slab and drop to replace those recommended in the report, assume in effect that the cantilever portion of a slab with a drop will be slightly larger than without a drop, but do not proceed to such unjustifiable lengths as the corresponding expressions in Equations (16) given in the report, which wholly ignore the redistribution of the constant total moment between mid-span and margin and the increase of that part of the unequalized moment distributed to any column due to decreasing the stiffness of the panel at mid-span.

In adopting Equations (19) for the thickness of the slab and drop, it is understood that the values of $d$ which are to be inserted, in order to obtain minimum values of $t$, are those obtained from Equation (12) and Table 2, although the foregoing criticism of Equations (16) is equally valid were we to adopt as correct the values of $d$ derived from Equation (8) and Table 1.

*2.—Flat Slabs as Statically Indeterminate Structures.*—In the second place, the writer objects to the adoption in the report of any theory of flat slabs as correct, which assumes them to be statically determinate structures, because they cannot be so regarded. The report, however, on page 1694, refers to the paper of John R. Nichols,* Jun. Am. Soc. C. E., as affording an approximate basis for its formulas. Now, any assumption of the validity and applicability of statical analysis to continuous flat slabs is incorrect, and leads to erroneous results, just as much as in the case of continuous beams or of any other indeterminate structures.

Any structure in which the magnitude or distribution of the stresses in any part of it undergoes any alteration by varying the rigidity of any of its members or elements is statically indeterminate. Hence, the principles of statics cannot be assumed to be applicable to such a structure unless there is definite proof that the statical principles sought to be applied are actually valid for the case in hand, so that it is beside the mark to adduce any statical limitations or requirements in flat slabs, such as are adduced in the report, because statical principles must here be subordinated to the principle of rigidities, which is the guiding principle in all indeterminate structures.

That a flat slab continuous over an unlimited array of separated supports is an indeterminate structure according to the above definition is evident from comparing the distribution of the stresses across the sides of the panels, due to the bending moments in a slab having most of the reinforcing steel concentrated in belts passing over the supports, with the distribution of the stresses across the sides of the

* "Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1670 (1914).

Mr. Eddy. panels where this is not the case, as it is not in the crown-sheet of a steam-boiler, for example. For, in the former case, the stresses across the sides due to the bending moments are largely concentrated at and near the supports, but it is far otherwise in the crown-sheet, which is supported on stay-bolts that pass through holes in the sheet, which completely destroy the radial resistance to bending at the supports. By reason of this distribution of the rigidity in the crown-sheet, the entire resistance to bending across any side of a panel is afforded by that part of the side lying between supports, and none of it is at the supports.

Statical principles, therefore, may be applied to flat slabs only after due inquiry and after it has been ascertained to what extent they may be applied to them. Statics and the principle of the lever are so ingrained in all reasoning about structures that the unconscious assumption of their applicability is likely to vitiate otherwise well-considered investigations of structures so complex in their interactions as indeterminate structures must necessarily be, and great care must be exercised in order to avoid the conclusions that inevitably follow from yielding to the insidious temptation unconsciously to make this assumption. The following unavoidable conclusion, therefore, may be formally stated:

> Proposition I.—Continuous flat slab floors on separated supports are indeterminate structures subject to the law of relative rigidities and its corollary, the law of least work.

In any indeterminate structure the relative rigidities of its parts control the distribution of the stresses in it strictly in proportion to the relative rigidities of the paths by which those stresses are carried, so that the greatest total stresses occur wherever the structure offers the greatest total resistance to deformation.

The steel reinforcement in a slab affords the principal resistance to tensile stress, and hence the total tensile resistance is, in general, where the cross-section of steel is greatest. Consequently, the distribution of the tensile stresses will in general be controlled by the quantity and position of the steel in the slab.

*3.—Shearing Stresses in Flat Slabs.*—Equilibrium requires that the total vertical shearing stress across the perimeter or margin of any panel shall be equal to the sum of the total live load resting on the panel added to the weight of the panel itself. The perimeter of the panel may be taken approximately at the sides of the rectangle the corners of which are at the four adjacent column centers, but it may be found somewhat more exactly by replacing parts of this rectangle near the columns by parts of the perimeters of the column capitals for 90° around each column center.

Shearing stresses transmit loads horizontally by the help of the bending moments which they bring into play, and they re-apply those loads

undiminished in total amount at the supports at some distance horizontally from their initial positions. Mr. Eddy.

In order to discuss this matter mathematically, designate the total uniformly distributed load on one panel by $W$, then, in case of a square panel loaded uniformly and supported at its four sides by walls or equal girders, the total vertical shear across a section close to one side is $\frac{1}{4}W$, as is evident either by symmetry or by the principle of the equal rigidities of the possible paths, lying in two directions at right angles to each other, by which the load may be carried to the supports.

Again, take the case of a single inside panel of any equally loaded panels supported on columns with square capitals, the edges of which are parallel to the sides of the panels; then, on the same principle, it is evident that the vertical shear across each side of each capital is $\frac{1}{4}W$. Now, whatever may be the shape of the perimeter of the capitals, the sections on which vertical shear occurs may be taken to be made up of an infinite number of successive elementary vertical faces all of which are parallel either to one or other of the sides of the panel. Then the total shear on all the vertical faces on one side of a capital, which are parallel to one side of a panel, is $\frac{1}{4}W$. This may be regarded as equivalent to saying that, whatever shape the perimeter of the capital may actually have, it can be assumed without material error for the purposes of computation to be replaced by a square capital.

It will be noticed that the intensity of the vertical shear across the perimeter of the panel is distributed in a very different manner, in the case of supporting walls on four sides, from its distribution across the edges of square capitals, in the case of supporting columns, for, in the latter case, the shear is all concentrated at the perimeter of the capitals and is zero at the other parts of the perimeter of the panel between capitals, although the total amount in either case must be equal to the panel load, $W$.

This concentration of shear is brought about by the bending and deformation of the slab, in which a shallow saucer-shaped hollow is produced around each panel center, while around each column center the slab is bent into the shape of an inverted saucer. The stresses accompanying these deformations effect the distribution of the shears just mentioned, as well as that of the bending moments also, a matter which will be treated later.

In case of wall supports on the sides, slab deformations are very different from this, as they produce the distribution of shears and moments that occur at the walls, which are very different from those at the perimeter of the capitals, a matter that has been treated by the writer elsewhere.* In all cases, the distribution of the vertical shears depends on, and is controlled by, the relative rigidities of the various

* "Concrete-Steel Construction", Eddy and Turner, p. 285.

Mr. Eddy. parts of the perimeter and supports, as expressed by their capacity to resist vertical forces without yielding. Any subsidence or yielding of a support or any part of it will alter the distribution of shears that otherwise would take place and reduce their intensity, and any inequalities in the resistance of the material at the perimeter of a capital will cause corresponding inequalities in distribution of the shears.

Again, consider a single square panel that is one of a single tier of equal square panels which constitute a wide beam and are supported on a succession of equi-distant transverse parallel walls or rigid girders across the tier. In this case no saucer-shaped hollows are produced by the load, for simple flexure alone occurs in such a case, and the deformation is designated as cylindrical, meaning thereby that the surface produced by flexure is a ruled surface such that any sections of it by vertical planes parallel to the wall supports are horizontal straight lines parallel to the walls. In this structure the entire shearing rigidity at the sides of a panel is at the walls, and is zero at the two unsupported sides. The total shear at each supported side is $\frac{1}{2}W$ and the total shear at any parallel vertical section between the walls will be twice that occurring at the corresponding section of an inside panel of a continuous slab of several tiers of panels on separated supports, such as was considered previously. As just stated, this shear of $\frac{1}{2}W$ is due to the fact that there are no supporting resistances or rigidities at the two unsupported sides of this panel.

This fundamental difference in the distribution of the total vertical shear at or across panel-wide sections in those two cases is of the utmost importance in the theory of flat slabs, because, owing to the fact already stated that a given panel load, $W$, can produce a total shear at the perimeter no greater than $W$, not only are the total shears in sections parallel to the sides of the panels of flat slabs on separated supports 50% less than in the wide beam structure just described, but, by reason of the necessary relation existing between the shears and the bending moments, the total applied bending moment produced at any panel-wide section of an inside panel is likewise reduced to 50% of that which would be produced if the total load were carried, as in the beam, by moments across that section and sections parallel to it, and none of it was carried by moments across sections at right angles thereto. Now, since in a continuous slab on separated supports equal rigidities occur in two sets of sections at right angles to each other, both must act at the same time, and each does 50% only of that which it would do in the absence of the other. In other words, since the shears in the sections parallel to either one of the sides of the panels in a continuous slab on separated supports are those that arise from a uniform panel load of $\frac{1}{2}W$, and equal shears also occur in sections at right angles thereto, it follows that the applied bending moments

across each of the set of panel-wide sections parallel to the sides of such a panel are those arising from a uniformly distributed panel load of only $\frac{1}{2}W$, instead of a load, $W$, as was the case in a beam. Mr. Eddy.

However, a wall panel in a flat slab floor that rests on a side-wall of the building at one side of the panel is prevented by the wall from having any deflection at that edge, so that the curvature of the panel under load is more nearly cylindrical near the wall than it would be with separated column supports at the edge. This makes the action of the panel more nearly like that of a wide beam with one end resting on the wall. In other words, the total shear in a section close to the wall will be greater than $\frac{1}{4}W$, but less than $\frac{1}{2}W$, while the total shear in a section at right angles to the wall and at the edge of the panel will be less than $\frac{1}{4}W$. The applied bending moments in these two directions in the panel will be such as to correspond to those shears.

An outside panel, however, the outer edge of which is supported on separated columns only, with no additional girder or stiffening along its outer edge, undergoes a greater deflection at the outer edge between supports than at other edges, and the total shear at a section near the outer edge is less than at corresponding sections near the other edges of the panel, and needs additional stiffening in the slab, either at the outside edge or parallel thereto. The effect of this kind of column support is to make the total shear in a section near the outer edge of the panel less than $\frac{1}{4}W$, while it makes that in a section of the panel at right angles thereto and near the side of the panel greater than $\frac{1}{4}W$. Corresponding changes occur in the total applied bending moments in the two directions in these panels. This last kind of outside panel with others contiguous thereto takes on more or less of the properties of a beam along the outside of the slab in which it differs materially from an outside panel resting on a wall, which last, it was seen, had some of the properties of a beam with an end at the wall.

A comparison of the two preceding paragraphs shows that outside panels supported at the edge of the slab on an outside row of columns may be designed so that their action will resemble closely that of the inside panels. In order that this may occur, the reinforcement at the outside edge must be such that at the heaviest loads to which it is subjected the deflections will be the same at mid-span between the columns at the edge as occur under these loads at mid-span between interior columns. These identical deformations will ensure nearly equal shears and bending moments in outside and inside panels, because the saucer-shaped deformations will be nearly the same in each.

*4.—Applied Bending Moments in Flat Slabs.*—It is a well-known proposition of beam theory that the sum total of half the numerical values of the applied negative bending moments at the ends of any

Mr. Eddy. span of length, $L$, plus the numerical value of the positive bending moment applied at mid-span, all arising from a uniformly distributed load, $W$, amounts to $\frac{WL}{8}$, whether the span is a simple one with end supports merely, or is continuous, or partly so, and this without regard to the loading or absence of loading on other spans of the beam. This proposition holds true regardless of the relative magnitudes of the moments of inertia of the beam at its successive cross-sections by which the relative rigidities of the beam in resisting the applied bending moments is expressed. It likewise is independent of the rigidity or lack of rigidity of the supports or their connections with the beam in resisting applied bending moments. The proposition, therefore, considers such a beam as an indeterminate structure to which the principle of rigidities applies in determining how the total moment, $\frac{WL}{8}$, is distributed between mid-span and ends. It will be convenient to designate this constant quantity, $\frac{WL}{8}$, as the total applied bending moment in the span, $L$, due to the uniform load, $W$. A proof of this well-known proposition may be found, among other places, in a paper by the writer.*

In accordance with the conclusions already reached in this discussion, the total applied bending moment in each direction parallel to the sides in a panel of a continuous slab on separated supports depends on the part of the load that is transmitted by the shears in that direction, so that, in an inside panel of such a slab, only one-half of the bending moment would be applied in each direction, and therefore the sum total of the numerical value of the positive applied bending moment across a mid-section parallel to one side added to the numerical value of the negative applied bending moment across that side is only $\frac{WL}{16}$. In any case, the sum, as above specified, of the applied bending moments in the two spans lying at right angles in any panel would together have the constant value, $\frac{WL}{8}$.

In view of this, and disregarding for the instant the effect of the size of the supports, we will state:

Proposition II.—In any square panel, inside or outside, of a continuous flat slab on rows of separated supports placed at successive distances of $L$ from each other and carrying a total uniformly distributed panel load, $W$, the sum total of the numerical values of the positive applied bending moments

* "A Further Discussion of the Steel Stresses in Flat Slab Floors", *Proceedings*, Am. Concrete Institute, 1916, p. 284.

acting across the two panel-wide mid-sections drawn at right angles parallel to the sides, added to half the sum total of the applied bending moments acting across the four sides of the panel, is equal to the constant quantity, $\frac{W L}{8}$. Mr. Eddy.

This proposition may be readily demonstrated for an inside panel in which the rigidities are equal and symmetrical about both of the mid-sections. For,

Let $S_x$ represent the total vertical shear across a panel-wide section at a distance, $x$, from the panel center;

and let $M_x$ represent the total applied bending moment at the same section.

Then, $dM_x = S_x dx$, for this equation expresses the necessary fundamental relation of shear to applied moment along $x$.

Similarly, for sections at right angles to this, the corresponding equation may be written:

$$dM_y = S_y dy.$$

Now, combining sections at right angles to each other and at equal distances from the panel center at which $x = y$, the sum of the two vertical shears across these sections is equal to the total load between either of them and the mid-section at $x = 0$, or at $y = 0$; that is, when $x = y$, we have $S_x + S_y = \frac{Wx}{L}$, in which $\frac{W}{L}$ is the total panel load per unit of width or length of panel.

In case of equal rigidities in directions at right angles, at equal distances, $x = y$, from mid-section, we have $S_x = S_y$, but not in case of unequal rigidities. In any case, at $x = y$ we have $S_x + S_y = \frac{Wx}{L}$.

Hence, by addition, at $x = y$,

$$d\,(M_x + M_y) = (S_x + S_y)\,d\,x = \frac{W x\,d\,x}{L},$$

and integrating between $x = 0$ and $x = \frac{1}{2}\,L$,

$$(M_x + M_y)_{x=y=\frac{1}{2}L} - (M_x + M_y)_{x=y=0} = \frac{W L}{8},$$

that is, the excess of the sum total of the positive moments across the two panel-wide mid-sections drawn at right angles to each other over the sum total of the negative moments across two sides mutually at right angles is $\frac{W L}{8}$. This excess, however, is the sum of the numerical values of these moments, which is the result stated in Proposition

Mr. Eddy. II. This holds true in general in case of unsymmetrical rigidities, although the demonstration is less simple for the more general case.

The distribution of the bending moment along each panel-wide section depends on the relative resistance to bending of the several parts of that section, as was pointed out, for example, where the bending resistance of the crown-sheet of a boiler was considered. The sub-division of the total applied bending moment, $\frac{WL}{8}$, between the two directions at right angles, however, depends not only on their relative resistances to bending moments at these sections, but also on their capacity to resist vertical shears as well.

As a result of the preceding investigation, we have consequently established the following:

Corollary.—In a square interior panel under a uniform load, $W$, the total numerical value of the positive bending moment at a mid-section, added to the total numerical value of the negative bending moment at an edge parallel to the mid-section, is only one-half of $\frac{WL}{8}$.

Disregarding for the instant the reduction of span due to the size of the capital, the report says:

"Analysis shows that * * * the numerical sum of the positive moment and the negative moment at the two sections named is given quite closely"

by $\frac{WL}{8}$, when written in the notation used herein. This shows that, for an inside panel, the analysis adopted in the report would make the applied moments twice as great as has just been shown really to exist. It is stated in the report, however, that the use of moment coefficients somewhat less than those derived by this analysis is believed to be warranted, and the reason assigned for this is the tensile resistance of the concrete. How small the allowance to be made on this account is, may be estimated from the recommended coefficients of $\frac{1}{25}$ at mid-span and $\frac{1}{15}$ at a margin, the sum of which is practically $\frac{1}{9.4}$ in place of $\frac{1}{8}$, as given by analysis. It is clear, therefore, that the recommendations in the report practically require provision to be made, so far as resisting applied moments are concerned, sufficient to carry the entire load by resistance in one direction only and then require, in addition, that provision shall also be made to carry the entire load by resistance in a section direction at right angles to the first, when, in fact, one-half must be carried in each direction. It is perhaps

unnecessary to remark that since moment magnitudes are of the nature of directed quantities, moments at right angles to each other are mutually independent of each other. Mr. Eddy.

*5.—Lines of Inflection.*—As ordinarily understood, lines of inflection or lines of contraflexure are those lines drawn on the slab at which successive panel-wide sections of the slab cut out by vertical planes parallel to the sides of the panel change curvature from convex to concave, or *vice versa.* It is evident that since there are two such sets of panel-wide sections at right angles to each other, there must be two sets of lines of inflection crossing each other. There would also be other lines of inflection for any other sets of sections, as, for example, sections parallel to the diagonals, etc. Considering, however, for the present, only those lines of inflection arising from panel-wide sections parallel to the sides, and in order to arrive at a more complete understanding of them and their relations to the deformations of the panels, let us take, for the sake of simplicity, the case of a homogeneous uniform flat plate, such, for example, as a steel plate of indefinite extent, resting on rows of equidistant separated supports in square array, and let it be subjected to equal concentrated loads, applied at the panel centers. Then, the upward resistance at each support is equal in magnitude to each of the concentrated loads. The top and bottom of the plate are then subjected to sets of forces which are alike in every particular except position and opposite direction, and the plate is bent in a perfectly symmetrical manner. On the surface of the plate draw lines parallel to the sides of the panels located half way between the supports and the panel centers. This will divide up the entire surface, checker-board fashion, into squares, the sides of which are one-half the distance between supports. These lines will be the inflection lines of the plate. The squares over the supports will be convex upward, those in which the panel centers lie will be convex downward, the others will be saddle-shaped. The bending moments across the lines of inflection will be zero.

If, now, the loading is supposed to be changed from concentrated loads to a uniformly distributed load, the curvatures in the central part of the panel will not be so sharp as in the case of concentrated loads, and the lines of inflection will thereby be removed slightly toward the supports, but will still remain straight. If, however, the plate is replaced by a slab which is reinforced so as to be stiffer over the supports than elsewhere, that will move those parts of the lines of inflection which are around the supports farther away from the supports, so that they no longer will be straight. If, in addition, the stiffness of the slab across the saddle-shaped areas is made less than elsewhere, those parts of the lines of inflection which lie around the panel centers will move nearer to the sides of the panel. In such a

Mr. Eddy. slab, the lines of inflection will divide up the area of the slab irregularly, in checker-board fashion, into areas which are only roughly rectangular, in which the convex and concave areas have bulging sides, but the saddle-shaped areas will have hollowing sides. The sides of these irregular areas are the *loci* of no bending across them along sections perpendicular to the sides of the panel, and, consequently, of no bending moment in that direction, but that does not necessarily signify that the stresses in either the steel or the concrete is zero across these lines. This is a fact, because the steel rods which are known to be in tension both in the tension areas around the supports and in those around the panel centers, are, no doubt, in tension throughout their entire lengths, not only in the tension areas, but across the lines of inflection as well, which fact would require the entire cross-section of the concrete to be in compression across lines of inflection, because, at any vertical section subjected to bending moment only, the sum of the tensions and compressions must be numerically equal.

From what has been already stated, it is evident that the location of the lines of inflection is fixed by the rigidity of the slab, and is largely within the control of the designer, so that it will be practically where the steel dips down below the neutral surface of the slab as it passes from the tensile zones at the top to those at the bottom of the slab.

Now, compare this with a design in which all the steel does not dip down at the same line. The effect of this would be to cause the lines of inflection to lie at one place for one concentration of loading and at another place for a different concentration.

The difference of action between these arrangements of steel is like that existing between the cantilever bridge and the continuous bridge of several spans. In the continuous bridge, the inflection point moves along the bridge to some extent as a train crosses, whereas, in the cantilever bridge, the point of zero moments is located definitely by a joint at the junction of the cantilever and suspended span. The same advantages are secured in flat slabs as in bridges by fixing the length of the cantilever as well as the relative size of cantilever head and suspended span by having all the steel dip down at a given line, for this arrangement reduces the bending resistance at the line so as to make it the line of inflection at all times.

It is evidently possible to make the cantilever extend out from the supports to any distance, even as far as mid-span, by sufficiently reinforcing the top of the cantilever throughout and, at the same time, reducing the resisting moment of inertia of the section at the edge of the cantilever. Now, this will be practically accomplished by making all rods dip together at the same line of inflection prede-

termined in such position as to separate the total applied bending moment of the panel into such parts as may be desired. This may be formally stated in the following: Mr. Eddy.

Proposition III.—In a continuous flat slab on separated supports the relative size of the cantilevers over the supports and the suspended spans within the panels, as well as the relative magnitudes of the total applied bending moments across the panel-wide sections at mid-span and at the sides of the panel, may be controlled by the location of the dip in the reinforcing rods across the neutral surface at predetermined lines of inflection.

Now, it is recommended in the report to have the steel dip down one part of it at one distance from the edge of the panel and another part at another distance, thus making the position of the lines of inflection absolutely impossible of determination for any loading, and different for different dispositions of load, and also making the subdivision of the total moment between mid-span and margin uncertain. The actual subdivision will then depend on several factors besides the positions where the steel dips down, such as the relative moments of inertia of the cross-sections at mid-span and margin, and the stiffness of columns, etc. The recommendation is bad in principle and more to be honored in the breach than in the observance, because making the rods dip down at different points renders bending moments indeterminate which, otherwise, would be determinate. This is discussed somewhat more in detail in the next section.

*6.—Column Rigidities and Relative Slab Rigidities across Panel-wide Sections, as Affecting the Sub-division between Mid-Span and Margins of the Total Applied Bending Moment in the Panel.*—In any beam of constant moment of inertia, fixed horizontally at the ends and uniformly loaded with a total load, $W$, the applied bending moment at mid-span is $\frac{W\,L}{24}$ and at each end is $-\frac{W\,L}{12}$. In case the moment of inertia or resistance to bending is made greater at and near the ends than elsewhere, the numerical value of the applied bending moment at each end will be increased at the expense of that at mid-span, but their numerical sum will still be $\frac{W\,L}{8}$. If, however, the restraint at the ends is reduced so as to be insufficient to fix them horizontally at the supports, the positive moment at mid-span will increase numerically, while the numerical value of the negative moment at each end will be decreased by the same amount. Similar principles apply to the moments at panel-wide sections of inside panels of a continuous slab over separated supports, except that the uniformly distributed load in the case of the slab, which causes the applied

Mr. Eddy. moment in each direction, will be only one-half of the total panel load, $W$, as previously shown. The column supports of a continuous slab are not usually sufficiently rigid to afford perfectly horizontal restraint at the supports, but there seem to be clear indications that, under ordinary circumstances, the columns resist about one-half the unequalized bending moments at the supports. That, however, is partly dependent on the rigidity of the connections between the slab and the supports.

In order to obtain an outside estimate of the divergence between the effect of the complete restraint at the ends previously considered and free tipping over the supports, let us suppose a uniform beam of three equal spans resting on four supports, with a uniform load on the middle span only, but free to tip at all four supports. The theorem of three moments shows that, in this case, the applied moments at the support would be $-\frac{W\,L}{20}$, while that at mid-span would be about $\frac{W\,L}{13}$. If, however, the beam were fixed horizontally as its two ends, but could tip freely over the two intermediate supports, the amounts over these would each be $-\frac{W\,L}{18}$, while that at mid-span would be about $\frac{W\,L}{14.4}$. These results, compared with those for the single span with fixed ends, show that although lack of fixity at the ends of the span, $L$, might reduce the numerical value of the applied moment by one-third or more at the edges of the panel, at the same time it might nearly double the moment, $\frac{W\,L}{24}$, at mid-span, even when no account is taken of any additional effect due to increased stiffness, such as is usual at the edges of the panel.

As previously shown, the foregoing results for beams should be divided by two, in order to make them applicable to inside panels of continuous slabs of equal moments of resistance at mid-span and margin.

How little the results thus obtained can be reconciled with the recommendation in the report which proposes to use the coefficient $\frac{1}{15}$, at the margin and $\frac{1}{25}$ at mid-span, regardless of relative column and slab rigidities, is left to the meditation of the reader.

*7.—The Distribution of Bending Moments Across Panel-wide Sections.*—As already stated, in connection with the bending of the crown-sheet of a boiler, the distribution of the rigidities in any panel-wide section completely determines and controls that of the intensities of the resisting moments in that section. Bending moments are by-

products and secondary phenomena in a beam or slab, and depend mathematically on the shear, which is, in fact, the primary action produced by the load. The rigidities which affect the distribution of shears are mostly independent of those which effect the distribution of the moments. The rigidities which control the distribution of the bending moments do this in two distinct ways, one is by the relative resistance across different parts of any panel-wide section, which determines the distribution and relative intensities of the stresses at the several parts of that section, the other is by the relative total resistances across the parallel sections at mid-span and margins of the panel, which determines the relative total moments across those sections. The effect of the latter has been considered previously. Mr. Eddy.

We are concerned here, therefore, with the sub-division of the total bending moment at one margin of a panel into two parts, one part across the two middle quarters, or inner section of the edge as it is called, and the other across the rest of the margin, or the column-head sections. The only possible basis for any sub-division or partition of the total marginal moment between these sections of the margin must be some existing or assumed distribution of the resistances across these sections. For, as already explained, the distribution of stresses will depend on the distribution of resistances. It should be noticed that, in case a distribution of stresses was found for a uniform plate by calculation, or by experiment, as was done by Trelease,* and then the slab was reinforced by belts of steel the resistances of which are made proportional to the stresses so found, the slab thus reinforced would no longer be one the rigidities of which were those of the original uniform plate, and the stresses in the slab would then be distributed in a manner very different from those in a uniform flat plate. So that any such basis for design is wholly illusory. This is also true regarding the subdivision of the total applied bending moment between the parts of the panel-wide section at mid-span.

The saddle-shaped deformations which occur at the sides of the panel directly between the column heads produce, in the top of the slab, tensions across the sides at mid-span and compressions parallel to the sides. These are stresses on vertical plane sections of the slab at right angles to each other, and express a state of stress in the material which may be expressed otherwise by saying that there are other vertical planes intermediate between these (inclined to them possibly at nearly 54°) on which the entire resultant stress is a shear horizontally along each plane, which increases in intensity according to its distance from mid-depth of slab.

The material of the slab effectively resists these shears, especially when it is well reinforced near mid-span of the sides, and such resist-

* "The Design of Concrete Flat Slabs", by F. J. Trelease, *Proceedings,* National Assoc. of Cement Users, Vol. VIII, 1912, p. 218.

Mr. Eddy. ance, called into play by the load, helps to carry the load, as do all other resistances, but this is a kind of resistance additional to any occurring in any simple beam structure, and it enhances the resistance of this part of the slab above that of simple moment resistance just as truly as the circumferential and radial action of the steel does in the convex and concave areas. It is by the help of these saddle-shaped deformations that the distribution of rigidities across the sides effect a corresponding distribution of stresses, as may be seen from the following discussion.

Suppose that little or no tensile resistance existed in the top of the slab across the edge of the panel near mid-span; then the panel on each side of that edge would deflect more readily for that reason, and the deflections at the panel centers on either side of this edge would be greater than otherwise. This sag on each side and yielding to bending across mid-span of the edge would transfer the applied bending moments each way from this point and bring them to bear on the slab at and near the column heads. If, however, very rigid reinforcement is introduced across the edge at and near mid-span, it will resist the greater portion of the total bending moment across the inner section of the edge and relieve the tensile stresses across the column heads by just so much, and so prevent any necessity for piling much steel directly over the columns. At the same time it will reduce the central panel deflections as well. Until it is proved that steel across the column heads resists bending moments more economically than across the inner sections, any recommendations such as those in the report, following the Chicago ruling, and intended to control the distribution of the steel across the margin, etc., are contrary to good policy and an intolerable restraint on legitimate design.

The truth of this last statement is evident from the following considerations. The intensities of compressive stresses across panel-wide sections must be distributed along those sections in a manner corresponding in the main with the distribution of the tensile stresses in the steel in these sections. In ordinary design, with steel belts massed across the supports, it has been frequently asserted that critical compressive stresses occur in the concrete around the column capitals. If this is true, it is due to the concentration of steel in the side belts, and can be obviated by such a disposition of the steel as to transfer a suitable fraction of the bending moments across the panel edges to the inner sections, and thereby relieve any excessive compressions in the outer sections. Any uncertainty which now exists respecting the existence of too large compressive stresses at right angles around the capitals can be thus removed, and the question of controlling the critical stresses in the concrete around the columns be thereby confined to the question of the vertical shearing stresses.

*8.—Bending Moments Reduced by Size of Capitals.*—In the next place, consider the distance by which the length of the span, $L$, between the column centers must be reduced in order to obtain a correct estimate of the effective span. How great such a reduction should be depends on several circumstances. In case the capitals are integral with the slab, and the columns perfectly rigid or nearly so, the effective span between capitals (each of diameter $= C$) would be $L - C$, and if all the reinforcing steel was to pass over these capitals, the length of the span would practically be $L - C$, while the panel load, $W$, would remain practically unchanged. Mr. Eddy.

As a mean value, under ordinary conditions of flexibility of columns, etc., the mean effective span may be conservatively taken as $L - \frac{2}{3} C$, although this may be subject to change in special cases. This may be stated explicitly as:

Proposition IV.—When a correction for the size of capitals is introduced, the constant applied moment in the span, $\frac{W L}{8}$, becomes approximately

$$\frac{W L \left(1 - \frac{2 C}{3 L}\right)}{8}.$$

If the panel, however, is one of a tier of panels supported on a series of successive transverse walls, each of width $= C$, instead of on separated supports, the load, $W$, would also need to be reduced in the same ratio as the span, and the constant applied moment per span thus corrected would become

$$\frac{1}{8} W L \left(1 - \frac{2 C}{3 L}\right)^2,$$

due to an effective span, $L\left(1 - \frac{2 C}{3 L}\right)$, and an effective panel load, $W\left(1 - \frac{2 C}{3 L}\right)$. It is evident, however, that this last result, which has been adopted in the report, is inapplicable to the case of the panels of a slab on separate supports.

Common practice makes $C$ not less than $0.2L$, while $C = 0.225$ is a usual value, and we have, consequently

$$0.87 > \left(1 - \frac{2 C}{3 L}\right) > 0.85 \text{ and } 0.76 > \left(1 - \frac{2 C}{3 L}\right)^2 > 0.72.$$

Hence, it appears that, by reason of the size of the capitals alone, the constant applied bending moment may possibly be reduced to

$$0.85 \frac{W L}{8}$$

Mr. Eddy. independently of any other reductions that may occur, such, for example, as that treated in the previous sections. However, the amount of reduction proposed in the report, which would make the total constant applied about $0.75 \frac{W L}{8}$, or about $\frac{W L}{11}$, is evidently unwarranted by theory, even in case the circumstances were such that the span were reduced by the entire diameter of the capital, for the minimum span would then be

$$L - C = L(1 - 0.225) = 0.775\,L.$$

9.—*Recapitulation.*—First.—The formula proposed in the report for the thickness of slabs does not conform to accepted usage in reinforced concrete design, which increases the steel ratio with increase in the ratio of thickness to span. Consequently, the formula for thickness makes the proposed minimum thickness too large for large loads and spans.

Second.—A continuous flat slab on separated supports is an indeterminate structure in which the distribution and relative intensities of the stresses depend on the relative rigidities of the parts acting, and, in particular, on the massing and location of the steel, so that the statical limitations applicable to ordinary statically determinate structures are not valid in such a slab.

Third.—On account of the necessary sub-division of the total shear arising from the panel load which is carried in the two directions parallel to the sides independently, the total applied bending moment acting parallel to each side is only 50% of that stated in the report in case of inside panels, and 60% of it in case of outside panels, provided the latter be taken to be 20% greater than the former, as recommended in the report.

Fourth.—The recommendation that the dip of the reinforcing rods from top to bottom of slab be distributed at various distances from the edges, is not well founded. A dip common to all neighboring rods should be used to fix the position of the lines of inflection and, at the same time, to fix the sub-division of the constant total applied moment between mid-span and margins.

Fifth.—The recommendation that the cross-section of steel across panel-wide sections at mid-span and margins be apportioned in certain proportions between the inner sections and the sections at the column heads, is without basis in sound theory, and should be cancelled.

Sixth.—The coefficient of reduction of the total applied moment due to the usual diameter of capitals is approximately $\left(1 - \frac{2\,C}{3\,L}\right) = 0.85$ and not $\left(1 - \frac{2\,C}{3\,L}\right)^2 = 0.75$, as recommended in the report.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

### Discussion.*

BY MESSRS. R. E. DOUGHERTY AND C. R. HULSART.

Mr. Dougherty.

R. E. DOUGHERTY,† M. AM. SOC. C. E.—The method used in driving this tunnel, which distinguishes it from the methods utilized in similar instances, involves primarily the so-called "pioneer tunnel." The arrangement is one which is somewhat new in tunnel driving, but the fact that the necessary results were secured by the contractors and the railroad company speaks for its success.

In sending out invitations to the contractors, the Chief Engineer of the Canadian Pacific Railway called attention to the fact that time was a very considerable factor, to such an extent as to be worth about $750 per day, and those who have had anything to do with railroad operation can very readily appreciate the significance of the time element. It is altogether possible that the tunnel might have been constructed by the so-called American methods, at a lesser cost, but the fact that all American records were broken, and the maximum results obtained by European methods approximated, would justify the means used.

Furthermore, from the standpoint of economy, reference might be made to an article by Mr. Sullivan, Chief Engineer of the Canadian Pacific, which appeared in one of the engineering periodicals, in which he states that the actual cost of construction was approximately

* Discussion of the paper by A. C. Dennis, M. Am. Soc. C. E., continued from April, 1917, *Proceedings*.

† New York City.

Mr. Dougherty.

$5.00 per cu. yd. of the main heading, from portal to portal, which included the cost of the pioneer tunnel, 5 miles of railroad to the site of the work, overhead charges, and all other costs involved. Furthermore, other bids secured by the railroad company ranged from $8.00 to $11.25 per cu. yd., although an estimate of $5.50 per cu. yd. was given by one contractor, based on the American method, but calling for a much greater length of time than was actually consumed. As a basis for comparison in the matter of progress, the maximum rate per month per heading attained at the Rogers Pass Tunnel was 946 lin. ft.; this approximates very closely the maximum rate of 1 013 lin. ft. per month attained in the construction of the Loetschberg Tunnel, in Switzerland.

In making a comparison of the methods used in the construction of other long tunnels, the only work which appears to have been on a similar basis is that involving the Simplon Tunnel in the Alps between Italy and France. The Simplon Tunnel was constructed for a single track, with a smaller auxiliary tunnel which was designed to become a part of a future parallel single-track tunnel, the prime purpose of the auxiliary tunnel having been to permit adequate drainage, as well as to facilitate the handling of material, etc. Without this auxiliary tunnel, it would have been almost impossible, in all probability, to construct certain portions of the Simplon Tunnel. In the case of the Rogers Pass Tunnel, however, the pioneer or auxiliary tunnel had no relation whatever to drainage, inasmuch as apparently ideal conditions were encountered. Furthermore, the position of the pioneer tunnel, with respect to the main heading, was such as to render it of comparatively small value for drainage purposes, had unfavorable conditions of that character been encountered.

A careful study of the paper and other literature appearing from time to time in engineering periodicals certainly justifies the conclusion that, although the method used was not, in all probability, the cheapest that could have been adopted, nevertheless, the results were unquestionably consistent with the combined elements of economy and time.

One feature to which attention is attracted is the fact that only 1½ miles of the tunnel will be lined. It would seem that this may be a possible source of trouble in later years, especially as experience in the eastern section of the country indicates that, in a number of instances, tunnels unlined at the time of construction have had to be widened and lined over traffic at an abnormal expense; and, even under the most favorable conditions, it would seem that experience justifies the excavation of a tunnel section of sufficient area to permit at least of later lining without the necessity of excavating over traffic.

Congratulations are heartily extended to the author for his contribution to the literature of the Society, and primarily for the part which he has taken in the work under discussion.

C. R. HULSART,* ASSOC. M. AM. SOC. C. E.—Milton H. Freeman, Assoc. M. Am. Soc. C. E., Resident Engineer on the East River Tunnels, joins the speaker in considering the pilot heading somewhat as a horizontal shaft. Of course, its length is greater than the average length of shafts in tunnels, but it is much cheaper per foot. To justify pilot headings on a purely economical basis would be rather difficult. They must be justified by their convenience and the facilities they afford for expediting the work. For instance, the Walkill Pressure Tunnel, one of the Catskill Aqueduct tunnels, more than 23 000 ft. long, had six shafts ranging from 330 to about 500 ft. deep. Of course, such a pilot heading would not be applicable to pressure aqueducts, but, taking that as an example, for those six shafts at a cost of $200 per ft. for the sinking and lining (not a bid price, but cost), such a drift could have been built for one-half the length of the tunnel, or, in other words, shafts about ⅜ or ½ mile apart could have been used, instead of ¾ mile apart. Mr. Hulsart.

Tunnel ventilation is an interesting subject. The wooden stave pipe gives a certain rigidity, which was lacking in the 12-in. galvanized-iron pipe used throughout the Catskill Aqueduct. It permits of the exhaust method, which most tunnel engineers generally favor. With the galvanized-iron pipe, the exhaust method is somewhat disastrous, because in exhausting, while blasting, the pipe frequently collapses. The speaker has seen several hundred feet of pipe collapse, and this the wooden stave pipe would not do. In ventilating the Newark Bay Tunnel on the Passaic Valley Sewer, R. H. Keays, Assoc. M. Am. Soc. C. E., used a 6-in. pipe, and blew air into the heading at a pressure of 7 lb. at the compressor, which gave excellent results. The ventilation was effective for more than 1 mile. The length of the tunnel was 11 000 ft., and one heading was considerably longer than 1 mile. The men were never overcome by smoke or fumes.

On the Catskill Aqueduct, when air had to be driven for some distance, with a fan which could create a pressure of perhaps only 7 oz., a relay method was sometimes adopted, with one or two fans in the tunnel for blowing in or exhausting.

The contractors for the Rogers Pass Tunnel made extensive use of compressed air, a plan which not many New York contractors would follow, to the same degree. Compressed air is a very expensive form of power, especially (as in this case) when compressed to 1 000 lb., for use in traction engines, trucks, and blowers. On the Walkill Pressure Tunnel, which was particularly well planned, some tests were made with air compressed to 100 lb. and used for pumping purposes. Considering the whole system, from the power that went into the motors operating the compressors, to the water pumped at the tunnels,

* New York City.

Mr. Hulsart. the efficiency was found to be about 7½%, and that of electric pumps, in small units, was about 55%; in other words, electricity was from seven to eight times as efficient as air compressed to 100 lb.

To compress air to 1 000 lb. requires two and a half times as much power, and it is used in any ordinary motor pump, hoist, or traction engine at a pressure of about 100 or 150 lb., so that the efficiency of such air-driven machinery would be about one-twentieth of that of electrically-driven machinery.

The Rogers Pass high-pressure plant required 1 500 cu. ft. at each end of the tunnel, or 3 000 cu. ft. of free air raised to a pressure of 1 000 lb. This involved the consumption of about 3 tons of coal per hour, during the time the two compressors were running (and it is assumed that they ran most of the time). Only one-twentieth of this fuel would have been required if electric power had been used for traction, power pumps, and blowers.

The shovel plates mentioned reminded one of the tendency, on a great many jobs, where everything is being sacrificed for speed, to forget to put them down, or perhaps eliminate them altogether. On one of the aqueduct jobs, where shovel plates were tried, the same gang mucked up the same quantity of rock in 30% less time with these plates than without them.

The cost of making records in driving in relatively small tunnels is worth studying. In the Catskill Aqueduct tunnel—equivalent to a single-track railway tunnel—records were sometimes rather expensive, as the progress in such a tunnel is limited to a great extent by the speed of mucking. If the speed is increased the muck is increased, more muckers are required in a smaller space, and the men are in one another's way. As an example of this, in one of the headings of the Walkill Pressure Tunnel, where the progress had been about 80 ft. per week, each man was handling about 2 cu. yd. of solid rock in 8 hours. The progress was increased to about 90 ft., but in order to do that, the number of muckers was increased, and each laborer handled about 1½ cu. yd. of solid rock in 8 hours. The progress was then increased to somewhat more than 100 ft., and each laborer handled a little more than 1 cu. yd. of solid rock in 8 hours. In view of these facts, another firm of contractors, carrying relatively small overhead cost, separated, as far as possible, the drilling and mucking operations. The firm lost sight of records to a large extent, and went in for economical tunnel driving. Three gangs of drillers and three gangs of muckers were used. While the muckers were at one heading, the drillers were at the other, and when they finished drilling and shooting one heading, they exchanged headings with the muckers. The mucking was very efficient. The speaker does not know how much rock each man handled, but, it was generally considered as a very economical job.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# PAPERS AND DISCUSSIONS



## THE RECONSTRUCTION OF THE STONY RIVER DAM

### Discussion*

BY MESSRS. WILLIAM CAIN, CHARLES E. GREGORY, KENNETH C. GRANT, L. R. JORGENSEN, EDWARD WEGMANN, IRVING P. CHURCH, AND M. M. O'SHAUGHNESSY.

WILLIAM CAIN,† M. AM. SOC. C. E. (by letter).‡—The writer is especially interested in the author's use of "anchoring walls" to increase the margin of security against the sliding of the dam. Such walls, projecting below the foundation, have been used repeatedly in dam and retaining wall design, but the projections were generally of small depth, so that little attempt has been made to state the principles affecting their design. The author's distinct contribution consists in the use of anchoring walls of such depth that the weight of the soil, from the foundation to the level of the lower end of the anchoring wall, is quite appreciable, and is utilized in increasing the resistance of the dam to sliding by lowering the possible plane of sliding from the foundation level to one passing through the bottom of the anchoring wall. He has entered into great detail regarding the principles affecting this design, and, in connection with it, has discussed the passive resistance of earth placed below the dam to sliding up a possible plane of rupture. Mr. Cain.

It is to be regretted that the author, in his experiments to determine the combined "adhesion" and friction of the various soils, did not apply increasing weights to the box containing the soil, and like-

* Discussion of the paper by F. W. Scheidenhelm, M. Am. Soc. C. E., continued from April, 1917, *Proceedings*.

† Chapel Hill, N. C.

‡ Received by the Secretary, April 2d, 1917.

Mr. Cain. wise measure the pull for each weight in turn, corresponding to "impending motion"; meaning by that term, that no actual sliding occurs, but that any increase in the pull, however small, would cause actual motion. If this had been done, the actual values of the coefficients of friction and cohesion could have been ascertained and used in the analysis. As the pulls were only recorded when the box containing the clay was in actual motion, the results give neither the friction coefficient alone nor the combined full cohesion and friction, such as would be actually exerted near the foundation of a stable dam, or perhaps on some lower plane or curved surface.

The results refer only to a combined friction and cohesion ("adhesion", as the author characterizes it, for this case), when the box was in motion. The numerical values of the "coefficients" in Table 2 are so much larger than the coefficients of friction of various clays, as given by Bell, as quoted in Table 7, that evidently a very appreciable amount of cohesion ("adhesion") was exerted during the motion. This negatives the idea, suggested by the writer in a previous paper, that, after motion began, possibly all cohesion was destroyed, so that only friction remained to resist the pull. Evidently, from the results, although a large part of the full cohesion of the solid clay was not exerted during the motion, still an appreciable part of cohesion in addition to the friction was exerted during the actual sliding of clay on clay.

Coulomb's laws, concerning "impending" sliding of earth on earth, are symbolized in the equation:

$$Q = f\, P_n + c\, A \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where $Q$ = the total resistance to sliding in the plane of shear, in pounds;

$f$ = the coefficient of friction;

$c$ = the cohesion in the plane of shear, in pounds per square foot;

$P_n$ = the normal pressure in the plane of shear, in pounds;

$A$ = the area, in square feet, of the plane where shear is impending.

Also, if $\phi$ = the angle of friction, then $f = \tan.\ \phi$.

Table 7 gives experimental values of $f$, $\phi$, and $c$, quoted from the writer's paper on "Cohesion in Earth"*, the first five values being from Bell, the last two from Jacquinot and Frontard.

The sand-clay, referred to in Table 7 was taken from a well-rolled dam, made of the best materials, its composition being 60% clay, 32% silica as an impalpable dust, and 8% silicious sand.

Mr. Scheidenhelm's Table 4 furnishes the figures given in Table 8.

* *Transactions*, Am. Soc. C. E., Vol. LXXX, 1916, pp. 1322, 1333.

TABLE 7.

Mr. Cain.

| Character of earth. | $f = \tan. \phi$. coefficient of friction. | $\phi$ | $c$, coefficient of cohesion, in pounds per square foot. |
|---|---|---|---|
| Very soft puddle clay, virgin state | 0.000 | 0° | 450 |
| Soft puddle clay, virgin state | 0.052 | 3° | 670 |
| Moderately firm clay, virgin state | 0.087 | 5° | 1 120 |
| Stiff clay, virgin state | 0.123 | 7° | 1 570 |
| Very stiff boulder clay, virgin state | 0.287 | 16° | 3 580 |
| Sand-clay, very humid, kneaded | 0.145 | 8°15′ | 385 |
| Sand-clay, damp, virgin state | 0.187 | 10°35′ | 443 |

TABLE 8.

| Character of soil. | Shearing value, in pounds per square inch of initial areas. | $c$, coefficient of cohesion, in pounds per square foot. |
|---|---|---|
| White clay | 12.1 | 1 742 |
| " " | 26.6 | 3 830 |
| " " | 20.7 | 2 981 |
| " " | 26.0 | 3 744 |
| " " | 22.8 | 3 283 |
| Black gumbo | 9.6 | 1 382 |
| " " | 16.1 | 2 318 |
| Black loam | 16.0 | 2 304 |
| Sandy yellow clay | 4.6 | 662 |
| " " " | 2.7 | 389 |
| Average | 15.7 | 2 260 |

Since, in the author's experiments on the direct shearing of clay, the normal pressure, $P_n$, on the initial area was zero, it is seen, from Equation (1), that the last two columns give the coefficient of cohesion, $c$, in pounds per square inch and pounds per square foot, respectively.

It is seen, also, that the values of $c$ compare very well with some of those in Table 7. Possibly, if the values of $\phi$ had been determined from appropriate experiments, they would also have compared favorably with the previous determinations.

It is important to observe that the time element affects all results very appreciably. Thus, Collin, in 1846, found that the resistance to shear in clay produced in from 12 to 15 min. was only one-third or one-fourth of that produced in from 12 to 15 sec. Bell states that all his tests were of considerable duration. This matter needs special attention by experimenters. As the author states that "the effect of the length of time of application of a given shearing load was not investigated," it is probable that the values of $c$, as given in Table 8, are all too large.

Mr. Cain. If certain values are assumed for $\phi$, from Bell's experiments on clays as roughly applicable to the author's experiments relating to the resistance of clay on clay when in motion, a rude surmise may be made of the corresponding value of $c$ for cohesion of motion.

Thus, for "white clay, fine-grained, wet", assume $\phi = 6°$; then, from Table 2, let $P_n = 200$ lb. and $Q = 0.34\ P_n = 68$ lb. With regard to $A$, the author says, "ordinarily, more than $\frac{1}{3}$ sq. ft. of the material under test was actually in contact." As definite information is lacking, it can only be assumed that $A = \frac{1}{3}$ sq. ft.; hence by Equation (1),

$$68 = 200 \tan. \phi + \frac{1}{3} c,$$

or, for $\phi = 6°$, assumed, tan. $\phi = 0.105$. Therefore, $c = 141$ lb. per sq. ft. For the same "white clay, moist", from Table 2, if $P_n = 200$ lb., as an average, $Q = 0.42\ P_n = 84$ lb. Therefore, for $\phi = 6°$ (as before) and $A = \frac{1}{3}$ sq. ft., by Equation (1), it is found that $c = 190$ lb. per sq. ft.

Again, for "yellow clay containing some grit, fairly wet", assume $\phi = 12°$ (tan. $\phi = 0.213$), $P_n = 200$ lb. Therefore, by Table 2, as an average, $Q = 0.68\ P_n = 136$. Taking $A = \frac{1}{3}$ sq. ft., by Equation (1), $c = 280$ lb. per sq. ft. Although the values of $A$ and $\phi$ assumed are doubtless incorrect, the roughly approximate values of $c$, as computed, are instructive as showing a very appreciable cohesion, even when the upper layer of clay is in motion.

The author gives, on p. 212,* for certain conditions pertaining to the original dam before failure, the horizontal water pressure = 1 013 600 lb.; weight of dam and water over it = 2 030 400 lb.; both for a 15-ft. bay. If the base is 52 ft. wide, $A = 52 \times 15 = 780$ sq. ft.

Consequently, assuming the low value, $c = 660$ lb. per sq. ft. (say, for the yellow clay) and $\phi = 12°$ (tan. $\phi = 0.213$), on substituting in Equation (1), we find,

$$Q = 2\,030\,400 \times 0.213 + 780 \times 660 = 947\,300 \text{ lb.},$$

which is but little below the water pressure tending to cause sliding. In fact, the resistance to sliding is exactly equal to the water pressure for $\phi = 13°\ 50'$, which is a possible value.

This computation involves the assumption that there exists a cohesion between the concrete and clay at the foundation, of 660 lb. per sq. ft., and the fact that the dam stood renders the assumption probable. No illustration can be given that more forcibly points to the need of "comprehensive experimentation" to determine the coefficients of cohesion and friction, not only for earth on earth, but likewise for concrete or other masonry on earth; for here is a dam that stood, which presumably should have failed if no cohesion was exerted at the base. This supposed cohesion that engineers (the writer included)

* *Proceedings*, Am. Soc. C. E., February, 1917.

have hitherto ignored, in computations affecting the sliding of dams and retaining walls, is probably a vital element concerning stability, and it doubtless has saved many walls from destruction by furnishing an additional resistance to sliding over that due to friction alone. Mr. Cain.

The argument that the resistance to sliding can be computed in terms of a friction coefficient alone, supposed to allow for the combined cohesion and true friction, is untenable, as the writer has conclusively shown.* The error can likewise be realized by a numerical example. Thus, on dividing Equation (1) by $A$, and putting $q = \frac{Q}{A}$ = unit total resistance to sliding, and $p_n = \frac{P_n}{A}$ = unit normal pressure, we have,

$$q = f\, p_n + c. \qquad \text{(2)}$$

From Table 8, the lowest value of $c$, for the "sandy yellow clay" is 389, say 400, lb. per sq. ft. On arbitrarily assuming $\phi = 12°$ or $f = \tan. \phi = 0.213$, Equation (2) reduces to,

$$q = 0.213\, p_n + 400.$$

Now, if it is assumed that a friction coefficient, $f = \frac{1}{3}$, with $c = 0$, can sufficiently represent the value of $q$, which denote by $q'$, we have, from Equation (2),

$$q' = \frac{1}{3}\, p_n.$$

It will be found, for $p_n = 3\,333$ lb. per sq. ft., that $q = q'$; but, for $p_n < 3\,333$, $q' < q$; whereas for $p_n > 3\,333$, $q' > q$.

Thus, for $p_n = 1\,000$ lb. per sq. ft., $q' = 333$, $q = 613$ lb. per sq. ft.; but when $p_n = 6\,000$ lb. per sq. ft., then $q' = 2\,000$ and $q = 1\,678$ lb. per sq. ft.

The differences would be much more marked had large values of $c$ been taken, corresponding to the "white clay" or "gumbo." As to the value $\phi = 12°$, arbitrarily assumed, it seems probable, from the experimental values of Bell, given in Table 7, that $\phi$ lies somewhere between 7° and 16°, and thus an average was taken as a probable value. Likewise, from the author's Table 2, the yellow clay shows a high, comparative value of the "coefficient of frictional resistance", including "adhesion", so that, possibly, $\phi$ should lie nearer to 16° than 7°, and 12° is possibly a little below the true value.

In connection with the foregoing values of $p_n$, it is of interest to know that, ignoring the cut-off wall and taking the area of the base of the dam as 52 × 15 sq. ft. for one 15-ft. bay, the average unit pressure on the foundation of the old dam is, $\frac{2\,030\,400}{52 \times 15} = 2\,600$ lb.

* "Cohesion in Earth", *Transactions*, Am. Soc. C. E., Vol. LXXX, pp. 1332–1334.

Mr. Cain. per sq. ft.; also, if the earth below it weighs 100 lb. per cu. ft., the unit pressure at 15 ft. below the foundation (at the bottom of the "anchoring wall", or at the level, *A B,* of Plate V), amounts to 4 100 lb. per sq. ft. The values of $q$ and $q'$, as found from the foregoing equations, do not differ greatly for these values, 2 600 and 4 100 for $p_n$, but this is only due to the particular values of $f$ and $\phi$ assumed.

It is not pretended that these are near the true values, as the computation was simply made to show what differences might occur by the two methods of estimating the resistance to sliding, and that the value of $p_n$, at the foundation or below it, is an important factor in the computation.

Although the data as to $c$ and $\phi$ are "not sufficient in number and reliability" to lead to an exact determination of the passive resistance to sliding of this dam, by the theory of coherent earth, nevertheless, some interesting conclusions can be drawn, by finding, by this theory, probable values of $c$ and $\phi$, necessary to the stability of the dam for each case of loading given by the author on pages 210-211.

It is very possible, as new experimental determinations come in, that surprises as to the true laws affecting friction and cohesion await us in the future, but, as far as the experiments go, Coulomb's two laws have been verified, and they will be made the basis of the following computations.

In the belief that the new method will prove of service, the analysis will be given in full for future reference, for immediate use by engineers who will determine experimentally the coefficients, $c$ and $f$, for any soil in question.

The computation will refer to the revised section of the dam, Plate V, but the cut-off wall will be ignored (which is on the side of safety), and the width of foundation (as estimated conservatively from the drawing) will be taken as 52 ft.

Thus, in Fig. 28, the width of foundation of the dam, $D E = 52$ ft.; the anchoring wall, or arm, extends from $D$ to $A$, 15 ft., the wall being just to the left of $A D$. The toe-wall extends downward from $E$ to $I$, 8 ft. To the right of this is earth, with the free surface, *E F M N.*

The analysis will proceed along the lines outlined by the author, but cohesion will be included and the resistance to sliding along $I J$ or $B C$ will be ignored. The problem then consists of finding the horizontal component of the resistance to sliding along planes such as $A I$ and $J N$ or along $A B$ and $C L$. Assuming the earth to weigh 100 lb. per cu. ft., the weight of earth, *A I E D,* is 59 800 lb., which, added to the weight of the dam and of the water on the deck, $2\,239\,400 \div 15 = 149\,293$ lb. per lin. ft. of dam crest, gives the weight on $A I \doteq 209\,000$ lb. Denote this weight by $G$, the length, $A I$, by $l$,

and its inclination to the horizontal by $\alpha$. If $N = a\,e$ is the normal reaction of the plane, $A\,I$ (making the angle, $\alpha$, with the vertical), then the resistance along the plane, acting to the left, to a horizontal force, $E$, tending to cause sliding along $A\,I$ to the right, is $c\,l + N\,f$. The friction component is $N\,f = \overline{de}$ and this, combined with $N = \overline{ea}$, gives a resultant, $\overline{da}$, making the angle, $\phi$, with the normal $\overline{ae}$, since $f = \tan.\ \phi$. Mr. Cain.

Hence, if we lay off, to scale, $\overline{ab} = G$, then draw $\overline{bg} = c\,l$ parallel to $I\,A$; from $g$, draw a horizontal, $\overline{gd}$, to the intersection, $d$, with the line from $a$, making the angle, $\phi$, above the normal $\overline{ae}$, then $\overline{gd} = E$, which is equal and opposed to the horizontal passive resistance to

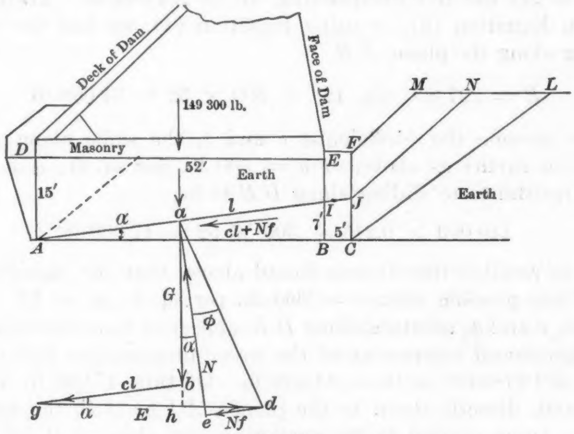


FIG. 28.

impending sliding up the plane, $A\,I$. This follows because the polygon of forces, $a\,b\,g\,d\,a$ or $a\,b\,g\,d\,e\,a$, is a closed polygon, where, $\overline{ab} = G$, $\overline{bg} = c\,l$, $\overline{gd} = E$, $\overline{de} = N\,f$, $\overline{ea} = N$, represent all the forces acting on the mass of earth, $A\,I\,E\,D$.

$E$ can be found from the diagram or from an equation easily deduced.

Thus, $E = g\,h + h\,d$

$= c\,l \cos.\ \alpha + (G + c\,l \sin.\ \alpha) \tan.\ (\alpha + \phi)$.

$$E = c\,.\,A\,B + (G + c\,.\,B\,I) \tan.\ (\alpha + \phi) \qquad (3)$$

It is evident from this equation,* regarding $G$ as the load on $A\,I$ for any position of $A\,I$ between the one shown in the figure and $A\,B$, that $E$ diminishes with $\alpha$ and is least for $\alpha = 0$, or along $A\,B$.

* On putting $(G + c\,.\,B\,I)$ in form $\left(227\,000 + A\,B \tan.\ \alpha \left(c - \frac{A\,B}{2}\right)\right)$, the weight of dam, water over it, and earth, $A\,B\,E\,D$, being 227 000 lb.

Mr. Cain. Now, to be conservative, take $\phi = 12°$ and $c = 300$ lb. per sq. ft., which is less than the smallest value (389 lb. per sq. ft.) given in Table 8 for the "yellow, sandy clay", and far less than the average, $c = 2\,260$ lb. per sq. ft. In fact, it is only slightly greater than the value (280) found for the yellow clay in motion. On substituting numerical values, $A\,B = 52$, $B\,I = 7$, $\alpha = 7°\ 30'$, $\phi = 12°$, $c = 300$, in Equation (3), we derive the resistance, exerted horizontally, to impending sliding up the plane, $A\,I$,

$$E = 90\,300 \text{ lb.}$$

To find the similar resistance to sliding along the horizontal plane, $A\,B$, first find the weight of earth, $A\,B\,E\,D = 78\,000$ lb., and add to 149 000 to get the $G$ corresponding, $G = 227\,000$ lb. Then, making $\alpha = 0$ in Equation (3), or using Equation (1), we find the resistance to sliding along the plane, $A\,B$,

$$E = 227\,000 \text{ tan. } 12° + 300 \times 52 = 64\,000 \text{ lb.}$$

If we assume the coefficients, $c$ and $f$, the same along $D\,E$ (for concrete on earth) as above, or $c = 300$ lb. per sq. ft., $\phi = 12°$, we find the resistance to sliding along $D\,E$ to be,

$$149\,000 \times 0.213 + 300 \times 52 = 47\,000 \text{ lb.}$$

If it is recalled that it was found above, that the stability of the old dam was possible when $c = 660$ lb. per sq. ft., $\phi = 13°\ 50'$, then the values, $c$ and $\phi$, assumed along $D\,E$, appear to be conservative.

The horizontal component of the water pressure per foot of length of crest is $1\,218\,200 \div 15 = 81\,000$ lb. Of this, 47 000 lb. is carried by the earth directly down to the planes, $A\,I$ or $A\,B$, the remainder, 34 000 lb., being carried to the anchoring arm along $A\,D$, by which it is finally transmitted to the earth to the right of $A\,D$. The average pressure on the earth along $A\,D$ is $34\,000 \div 15 = 2\,267$ lb. per sq. ft., which is perhaps not so great as to cause flow, though the exact distribution of stress along $A\,D$ is unknown.

It is well, however, to enquire if this pressure of the arm, $A\,D$, may not cause sliding of the earth up some plane through $A$, shown by the dotted line. Hence the passive resistance of the earth to sliding up such a plane will next be investigated. Assuming that the reaction of the anchoring wall on $A\,D$ is horizontal, the unit horizontal passive resistance, $a$, of the earth at any point of $A\,D$ can be found from the equation,*

$$a = b \text{ tan.}^2 \left(45° + \frac{\phi}{2}\right) + 2\,c \text{ tan.} \left(45° + \frac{\phi}{2}\right);$$

where $b$ = the weight, in pounds per square foot, acting on a horizontal plane at the point considered. As the distribution of stress on $D\,E$ is

* Cain's "Earth Pressure", p. 192.

not given, assume it to be uniform, so that at $D$, $b = 149\,300 \div 52 =$ 2 870 lb. per sq. ft.; and at $A$, $b = 2\,870 + 15 \times 100 = 4\,370$ lb. per sq. ft. Hence, taking, as before, $\phi = 12°$, $c = 300$ lb. per sq. ft., the equation gives the unit passive resistance, acting horizontally to the left, Mr. Cain.

at $D$: 5 120 lb. per sq. ft.;
at $A$: 7 400 lb. per sq. ft.

As the flow of the earth is supposed to occur at 4 000 lb. per sq. ft., it is seen that sliding up the dotted plane will not occur when the maximum permissible stress on $AD$ is taken at 4 000 lb. per sq. ft., as given by the author. If the permissible value to prevent flow had been assumed to be greater than 7 400 lb. per sq. ft., then the maximum unit compression at $A$ must be taken at 7 400 lb. per sq. ft. or less; otherwise the earth at $A$ would tend to slide up the dotted plane, which makes an angle, $45° + \frac{\phi}{2} = 51°$, with the vertical.

It now remains to compute the passive resistance, exerted on $F J$ or $F C$, of the earth to sliding up the proper planes of rupture, $J N$ or $C L$, corresponding. The graphical construction for effecting this will be given presently; but, to complete the present investigation, the result may be anticipated that the passive resistance on either $F J$ or $F C$ exceeds the safe compressive stress of the earth, so that the latter will limit the available resistance. The unit passive resistance at the free surface is not zero, but a very appreciable quantity, as will be shown eventually, and reasons will be given why an average pressure on $F J$ or $F C$ of 3 000 lb. per sq. ft. can be allowed without fear of the compressive strength of the earth being exceeded, or flow being imminent. Thus, the total compression, acting horizontally, that will be allowed on $F J$, 9 ft. in depth, is,

$$3\,000 \times 9 = 27\,000 \text{ lb.},$$

and on $F C$, 16 ft. in depth,

$$3\,000 \times 16 = 48\,000 \text{ lb.}$$

It has been shown previously, omitting any resistances along $I J$ and $B C$, that the horizontal component of the resistance to sliding along $A C$ is 64 000 lb., and along $A J$, 90 000 lb.; hence the total horizontal component of the resistance,

along $A C$ and $C F = 64\,000 + 48\,000 = 112\,000$ lb.
and along $A J$ and $J F = 90\,000 + 27\,000 = 117\,000$ lb.

Both of these totals exceed the horizontal component of the water pressure, 81 000 lb.; so that, if the assumed values of, $c = 300$ lb. per sq. ft. and $\phi = 12°$, are safe values, the dam is secure against sliding, for the conditions assumed, along the planes taken. The conclusion will not be altered if any plane intermediate between $A J$ and $A C$ is taken in place of either of these planes. From what precedes,

Mr. Cain. it seems that $c = 300$ lb. per sq. ft. is a safe value, but there is uncertainty as to the value of $\phi$. If we assume $\phi = 4° 23'$ (tan. $\phi = 0.0767 = f$), it will be found that, using $c = 300$, as before, the horizontal component of the water pressure is exactly equal to the resistance exerted along $A\,C$ and $C\,F$. For a less value of $\phi$, sliding would occur. From Table 7, the value $\phi = 4° 23'$ is seen to be a barely possible one, though the probable value is greater; hence, with the incomplete data as to $c$ and $\phi$, it can only be asserted, for the loading of the author's Case I, that this investigation shows that the dam is most probably safe against sliding. The investigation for the loading under Case II, or "under the most severe conditions within limits of reason", page 211, is exactly similar to the foregoing, so that only results will be given.

The uplift water pressure was subtracted from the former load on the foundation, giving a net load, per foot of crest, on the foundation of 90 400 lb. Of course, this uplift pressure must not be taken in addition from the weights, $A\,I\,E\,D$ or $A\,B\,E\,D$, of earth, since, from the unit water pressure (say) just to the right of $A$, acting upward, is to be subtracted the weight of a column of water of height, $A\,D$, and horizontal section unity, which gives, precisely, the uplift at the level, $D\,E$; so that the weight of earth, $A\,I\,E\,D$ or $A\,B\,E\,D$, is not to be increased by the weight of water above $A\,I$ or $A\,B$, or diminished in any way, since the uplift pressure has already been subtracted from the load on $D\,E$. The total horizontal force acting on the dam, due to water pressure and ice, is 101 300 lb. per lin. ft.

By an investigation similar to the foregoing, it is found, for $c = 300$ lb. per sq. ft., $\phi = 12°$ and the compression on $C\,F$ limited to 3 000 lb. per sq. ft., that the total horizontal resistance along $A\,C$ and $C\,F$ amounts to 99 400 lb., and along $A\,J$ and $J\,F$ to 96 500 lb. However, for $c = 388$ lb. per sq. ft., $\phi = 12°$, the least resistance is equal to the water and ice pressure, 101 300 lb.; so that the dam is secure against sliding for either Case I or Case II, if $c = 390$ lb. per sq. ft. and $\phi = 12°$.

It is interesting to know the amount of security when $c = 660$ lb. per sq. ft., $\phi = 12°$. It is found, for these values, that the horizontal resistance exerted along $A\,B$ and $C\,F$ amounts to 118 000 lb., and along $A\,I$ and $J\,F$, to 116 000 lb. The latter value exceeds the water and ice pressure by 15 per cent. Since it was found above that the old dam was just stable for $c = 660$ lb. per sq. ft., $\phi = 13° 50'$, the values assumed, $c = 660$, $\phi = 12°$, seem to be not unreasonable. Again, for $c = 660$, $\phi = 12°$, the horizontal resistance to sliding along the foundation is, for Case II, $90\,400 \times 0.243 + 660 \times 52 = 53\,600$ lb.; hence the anchoring wall at $A\,D$ carries,

$$101\,300 - 53\,600 = 47\,700 \text{ lb.}$$

or, $47\,700 \div 15 = 3\,180$ lb. per sq. ft., average pressure.

As stated above, the resistance along $BC$ (5 ft. in length), as well as that along $IJ$, was neglected, there being but little weight over parts of these surfaces; also, the cut-off wall was omitted, or it was supposed to be cracked at $A$. There is evidently some resistance from the omitted portions. Mr. Cain.

It must be distinctly understood that, although probable values for $c$ and $f$ for Cases I and II, have been found above, it is not asserted that these are the real values exerted. More extended experimental data as to these coefficients are required before definite results can be predicted.

It remains to find the passive resistance of the earth to the right of $C F$ or $J F$ to sliding up some plane, as $C L$ or $J N$. Although the assumption of a plane surface of rupture, as $A B$ or $A I$, for the earth under the dam, is doubtless near the truth, the same cannot be said for the earth below the dam, of the irregular free surface, $F M L$. Theory shows that, except when the free surface is level, the surface of rupture is curved; so that when it is, of necessity, assumed to be plane, in the graphical construction that follows, the results must be treated as only an approximation to the truth, and a factor of safety must be used. Here, again, no experiments as to passive resistance have been made, so that we are absolutely in the dark as to the true surface of rupture and the resulting passive thrust.

The graphical construction alluded to is given in Fig. 29, where $A B = 16$ ft., $B g = 11.5$ ft., and $\overline{go} = 11.5$ ft.; $A B$ ($= C F$ of Fig. 28) being vertical and $\overline{0\,5}$ ($= M L$ of Fig. 28) being horizontal. It is desired to know the least force, acting horizontally to the right on $A B$, that will cause sliding up some plane of rupture $A\,4$. Draw a number of trial planes of rupture, $A\,0$, $A\,1$, . . ., and draw $A\,a$ horizontal. Then, with $A$ and some point, $g$, on the vertical, $A B$, produced, as centers, and with the same radius, $A\,g$, describe two arcs of circles, $a\,a_0$ and $A\,s\,s_0$. Assuming $\phi = 12°$, lay off $A\,g\,s = \phi = 12°$ and, with dividers, lay off the chord, $s\,s_0 =$ the chord, $a\,a_0$, the chord, $s\,s_1, =$ the chord, $a\,a_1$, etc., hence we have the equality of the angles, $s\,g\,s_0 = a\,A\,a_0$, $s\,g\,s_1, = a\,A\,a_1$, $s\,g\,s_2 = a\,A\,a_2$, etc. Then $g\,s_0$, $g\,s_1$, $g\,s_2$, . . . , make the angles, $\phi$, above the normals to the planes, $A\,0$, $A\,1$, $A\,2$, . . . , respectively. This is easily seen by referring to the force diagram of Fig. 28, in which it is observed that $a\,d$ makes the angle, $\phi + \alpha$, with the vertical. Similarly, in Fig. 29, if (say), the angle $a\,A\,4 = \alpha$, then $A\,g\,s_4 = \phi + \alpha$, since we laid off $s\,g\,s_4 = a\,A\,a_4 = \alpha$; hence, as in the case of $a\,d$ in Fig. 28, $g\,s_4$ makes the angle, $\phi$, above the normal to the plane, $A\,4$. It is convenient here to lay off the angle $\phi = A\,g\,s$ first, and then lay off the successive $\alpha$'s above $g\,s$.

Let it be assumed that the earth weighs 100 lb. per cu. ft. and that the coefficient of cohesion is $c = 300$ lb. per sq. ft.

Mr. Cain.

The next step is to compute the weight of the successive trial prisms of rupture, $A\,B\,0$, $A\,B\,0\,1$, $A\,B\,0\,2$, . . . , and lay off to scale on the vertical, $g\,A$, $g\,g_0$, $g\,g_1$, $g\,g_2$, . . . , to represent these successive weights. If, on the horizontal, $g\,0\,4$, we construct $\overline{0\,1} = \overline{1\,2} = \overline{2\,3}$, . . ., the triangles, $A\,0\,1$, $A\,1\,2$, . . . , are all equal in area, so that the total areas can be computed quickly by successive additions. The thickness of the prisms perpendicular to the plane of the paper is assumed as unity, so that areas and volumes are represented by the same numbers.

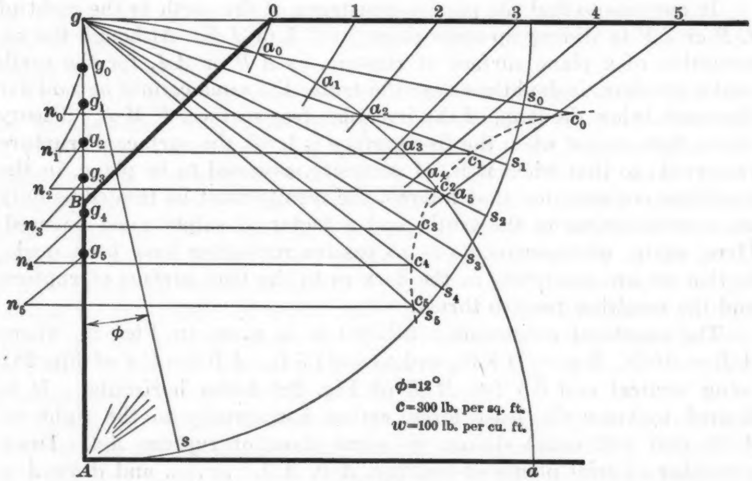


FIG. 29.

We next measure the lengths, $A\,0$, $A\,1$, $A\,2$, . . . , to the scale of distance, and multiply the lengths, in feet, by $c = 300$, to find the successive cohesive forces acting downward (opposed to impending motion) along the planes, $A\,0$, $A\,1$, . . . , respectively. Lay off, to the scale of loads, $g_0\,n_0$, $g_1\,n_1$, . . ., parallel, respectively, to $A\,0$, $A\,1$, . . . , and equal to the cohesive forces acting along these planes. From $n_0$, $n_1$, . . ., draw horizontals to intersections, $c_0$, $c_1$, . . ., with $gs_0$, $gs_1$, . . .; then the smallest of the lengths, $n\,c$, to the scale of loads, will give the passive earth thrust on $A\,B$. In this figure, $n_4\,c_4$ is the shortest of the lines, $n\,c$, and it measures to the scale of loads, 69 300 lb., which is the passive thrust required, and $A\,4$ is the corresponding plane of rupture. The method, just outlined, consists in treating in turn, $A\,B\,0$, $A\,B\,0\,1$, . . . , as trial prisms of rupture and constructing the successive polygons of forces, $g\,g_0\,n_0\,c_0\,g$, $g\,g_1\,n_1\,c_1\,g$, . . ., similar to the polygon, $a\,b\,g\,d\,a$, of Fig. 28. The poly-

gon, $g\ g_4\ n_4\ c_4\ g$, is found to have the least value of the thrust, $n_4\ c_4$, hence motion is impending for this thrust on $A\ B$, only along the plane $A\ 4$, since a greater thrust on $A\ B$, acting from left to right, is required to cause impending motion up the other trial planes of rupture. The forces in equilibrium acting on the true and only prism of rupture, $A\ B\ 0\ 4$, are, to scale, $g\ g_4$ = weight, in pounds, of $A\ B\ 0\ 4$, $g_4\ c_4$ = cohesion on $A\ 4$, acting down and parallel to $A\ 4$, $n_4\ c_4$ = thrust on $A\ B$, $c_4\ g$ = resultant of $N$ and $N$ tan. $\phi$, where $N$ = normal reaction of the plane, $A\ 4$, and $N$ tan. $\phi$ is the friction acting down (opposed to impending motion) along the plane, $A\ 4$. The resultant of $N$ and $N$ tan. $\phi$, thus makes the angle, $\phi$, above the normal to $A\ 4$, as constructed in the beginning. Mr. Cain.

For the thrust, $n_4\ c_4$, on $A\ B$, acting from left to right, motion is impending not only along the plane, $A\ 4$, but upward along the plane, $A\ B$; hence, strictly, cohesion and friction are exerted downward on the plane, $A\ B$. Neglecting the cohesion, this would require that the lines of the type, $n\ c$, should be inclined below the normal to $A\ B$, or below the horizontal, at the angle, $\phi$. Hence intersections, as $c_4$, will move down the corresponding $g\ s_4$, so that $n_4\ c_4$ and its horizontal component will be greater than before. The same is true for all the trial thrusts, of which the least is to be taken as the true one. The horizontal component of the true thrust, thus found, is greater than that found before, where the pressure on $A\ B$ was supposed to act horizontally.

From lack of any experimental data concerning passive thrust, it seems best, at present, to take the smaller thrust, corresponding to a horizontal pressure on $A\ B$, since the surface of rupture is curved and the assumption made that it is plane may possibly give results far from the truth. Hence, in computing the various values of the passive resistance to the push of the earth to the left of $C\ F$, Fig. 28, acting to the right, the push or thrust was taken as acting horizontally, as in Fig. 29.* For accuracy, other planes, lying near $A4$, should be tested in order to ascertain the minimum thrust represented by a line of the type, $n\ c$, or calculation can be resorted to. In fact, Equation (3), referring to Fig. 28, is directly applicable to each trial prism of rupture, provided $G$ represents its weight.

* It may occur to one who has read pages 180–181 of the writer's "Earth Pressure", that, for the slope, $FM$, of Fig. 28, the theory pertaining to the uniform slope of indefinite extent, may be applicable; but, since $FM$ is of limited extent, and the pushing force exerted on $CF$, from left to right, does not act parallel to $FM$, the conditions postulated for this theory do not obtain, and the theory is inapplicable. The surface, $FM$ makes an angle, $i = 45°$, with the horizontal, and if it was of indefinite extent, below and above $F$, slipping would occur at a depth, $x_0 = 7.62$ ft., when $\phi = 12°$, $c = 300$ lb. per sq. ft. by the theory alluded to, and the passive thrust, acting parallel to the surface, would be much smaller than that found above. If $FM$ was large, say, 100 ft., at the same point of the slope, the theory might be approximately applicable; but, even then, the only effect would be a readjustment of the free surface contour to a new surface consistent with equilibrium.

Mr. Cain.

Thus, in Fig. 30, let,

$G =$ the weight of the trial prism of rupture $A\,D\,M\,I$;

$x = A\,B =$ the horizontal projection of $A\,I$;

$y = B\,I =$ the vertical projection of $A\,I$;

$\alpha =$ the angle, $B\,A\,I$;

whence tan. $\alpha = \dfrac{y}{x}$.

Then Equation (3) takes the form,

$$E = c\,x + (G + c\,y) \text{ tan. } (\alpha + \phi) \ldots\ldots\ldots\ldots (4)$$

By the use of this equation, it is not necessary to draw Fig. 29 at all, as the equation can be applied to each trial prism of rupture in turn. Thus, for $\phi = 12°$, $c = 300$, for the plane, $A4$, of Fig. 29, $x = 31.5'$, $y = 27.5'$; therefore tan. $\alpha = \dfrac{y}{x} = 0.873$; therefore $\alpha = 41°\ 07'$; also $G =$ the weight of $A\ B\ 0\ 3 = 36\,700$ lb.; whence by Equation (4), $E = 69\,400$ lb.

Similarly, for the plane, $A3$, $x = 26.5$, $y = 27.5$, whence $\alpha = 46°\ 8'$; also $G =$ weight of $A\ B\ 0\ 3 = 29\,863$, and finally, $E = 69\,250$ lb. For a trial plane of rupture from $A$ to a point midway between Equations (3) and (4), it is similarly found that $E = 69\,100$ lb.; hence this plane can be regarded as the true plane of rupture, and the thrust, 69 100 lb. (the least of the three), as very nearly the true thrust.

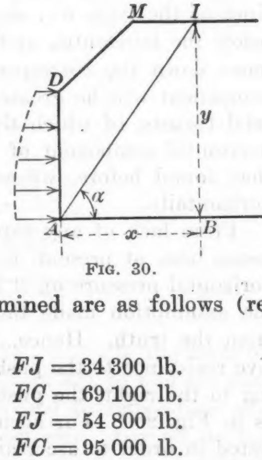

FIG. 30.

The passive thrusts for all the cases examined are as follows (referring to Fig. 28):

$c = 300$, $\phi = 12°$, passive thrust on $FJ = 34\,300$ lb.
" , " , " " $FC = 69\,100$ lb.
$c = 660$, $\phi = 12°$, " " $FJ = 54\,800$ lb.
" , " , " " $FC = 95\,000$ lb.

In the first case, $N$ is to the left of $M$; for the other cases, both $N$ and $L$ were found to be to the right of $M$. These thrusts exceed the compressive strength of the material along $FJ$ or $FC$, and hence they cannot be utilized in estimating the resistance of the dam against sliding, as previously noted.

It has been stated previously that the unit passive thrust at the free surface of the earth was not zero. To find approximately its amount at the point, $D$, let us find the thrust on a vertical plane extending 1 ft. below $D$, Fig. 30, by repeated use of Equation (4). The minimum value of $E$ which is the true thrust, is found to be 2 700 lb. per sq. ft. It corresponds to $\alpha = 61°$, nearly. By consider-

ation of the total thrusts on the areas, *FJ* and *FC* of Fig. 28, it was estimated that the unit thrust at *C* was about double that at *F*. The unit thrust is not uniformly increasing, but varies about as shown by the little arrows in Fig. 30. Mr. Cain.

The unit compression on *FC*, Fig. 28, can be supposed to follow the same law, though, for simplicity, it can be regarded as uniformly increasing from a stress of 2 000 lb. per sq. ft. at *F* to double this, or the maximum allowable, 4 000 lb. per sq. ft., at *C*. This gives an allowable average stress on *FC* of 3 000 lb. per sq. ft., as used above.

It is seen from this investigation that the author is warranted in using a larger average safe stress on *FC* than the specified 2 000 lb. per sq. ft.

It may be observed that, if the force tending to cause sliding of the dam, is entirely resisted along the plane, *AB*, then only the active pressure of the earth to the right of *FC* is exerted. If then, the resistance along *AB* is supposed to diminish, first the active thrust of the mass, *CFML*, will be overcome and then more or less of the full passive thrust of this mass will be exerted, the amount always being less than that which would exceed the safe compressive stress of the earth, or would cause flow of the material along *CF*. Consequently, it does not appear that the passive resistance of any pile of earth placed below a dam to help resist sliding will ever come into play unless there is a slight movement of the dam down stream, since earth is not a rigid body.

A year or so ago, the writer's attention was called to the case of a high dam, where a large pile of earth was placed below and against the dam, to prevent possible sliding. He stated then that the coefficients of friction and cohesion were both needed to investigate fully the extra resistance to sliding supplied by the earth. He is glad of this opportunity to offer the full solution for such cases. Whatever the free surface contour, the graphical method of Fig. 29 offers a quick, approximate solution, though it is well, for the reasons stated, to apply a factor of safety to the result, or otherwise to use conservative values of *c* and *f*.

No better illustration could be given of the need of "comprehensive experimentation to determine the coefficients of friction and cohesion," than is afforded by this constantly recurring problem of estimating the resistance to sliding of dams and retaining walls. The writer is firmly convinced that a correct solution can only be attained by the use of the theory of coherent earth.

In conclusion, the writer wishes to state his appreciation of the very thorough and painstaking manner—down to the minutest details—in which the author has done the work of reconstructing the Stony River Dam.

Mr. Gregory. CHARLES E. GREGORY,* ASSOC. M. AM. SOC. C. E.—A paper of this kind is unusually interesting, because it deals with the failure of a structure. The speaker believes that one can always learn very much more from the failure of a structure than from any discussion of what one believes ought to happen in accordance with theories.

The author has given a most lucid and well-written description of how he corrected the mistakes of the first dam and has most ingeniously met the difficulties of the problem while utilizing the portions of the old dam which remained after the failure.

The reconstruction of this dam is an interesting subject, because it deals with a number of uncertainties pertaining to both the site and the type of dam, and uncertainties are always interesting. About the only relatively definite elements in the whole problem are the strength of concrete and steel. Nearly everything else is open to more or less uncertainty. Notwithstanding this, the speaker believes all will admit that any large dam should be built so that its safety cannot be questioned, and one cannot say that any structure is safe until the uncertainties have been eliminated.

In this case the type of dam and the site having been determined, there remain a great number of very uncertain features to be considered. The author has pointed out most of them, and has told how they were removed.

In any hollow dam, the uplift is the great doubtful feature and the great enemy of a dam of this type. There is no way of knowing to a certainty just what the uplift pressure is and how it is distributed. In designing a dam of this type, certain assumptions are made as to this pressure. It is assumed that if uplift exists, it can be controlled and limited by the weep-holes. Therefore, numerous weep-holes are placed in the foundation, and are supposed to relieve the upward pressure, and limit it to a relatively small amount. The cut-off wall must be effective, and must cut off the water from the impounded reservoir.

It is common experience that it is exceedingly difficult to build a cut-off wall, of any great depth and length, which will be absolutely water-tight. A concrete wall of great length will shrink and crack, in spite of anything that can be done to prevent it. Construction joints form weak planes, if not actual joints. All these conditions are favorable to leakage, even when one is successful in securing impervious rock foundation. Some water is bound to get through, and then it is bound also to show itself under the bottom of the dam.

When weep-holes are provided and are effective, they will allow this water to escape and relieve the pressure, but if they are not effective, the upward pressure will obtain. If they are effective, however, the water flowing through the soil under the bottom of the dam is a new source of danger.

* Mt. Kisco, N. Y.

A safe structure should not be menaced by flotation or by undermining. The flowing water from the weep-holes is certain to carry with it more or less soil, especially if it consists of fine, light grains; and undermining of the bottom of the dam will gradually occur. Eventually, the flow will become greater, heavier grains of soil will be carried, and ultimately there will be sufficient undermining to cause settlement and cracking, increasing the upward pressure so as to cause the dam to overturn or slide. Mr. Gregory.

Probably this dam started to fail by overturning due to undermining. As soon as the tendency to overturn developed tension at the upper toe sufficient to open a joint and admit the water, the upward pressure floated the whole section off its base. The failure was probably a combination of overturning and sliding.

The speaker has not made any check computations whatever of the structure, but from the author's statement of the various forces acting on this dam it would seem that, unless high coefficients of friction are assumed for the base, it would not be safe from sliding. There seems to be no good ground for assuming high coefficients for a soil which is saturated with upward flowing water to the weep-holes.

In the speaker's opinion, the initial mistake was made in selecting a dam of this type for such a site. Although the speaker has not visited the site, and knows nothing about it, except what is disclosed in the paper, there appears to be in the vicinity considerable clay, gravel, and sandy clay which would be excellent material for an earth dam. An earth dam could have been built on this site economically, and would have eliminated nearly all the very uncertain features which obtain with this hollow concrete dam; the speaker believes an earth dam is the proper type for this site.

Another interesting part of the paper is the discussion of the spillway capacity. The author certainly has increased the capacity of the original spillway to a point beyond all precedent. The speaker believes that any spillway should have a very safe capacity. It should be large enough to care for the very largest storm that can possibly be conceived of, but the speaker believes that, in this case, the author has surely gone beyond the necessary limit. A flood as great as that provided for by the new spillway certainly could be produced only by a rainfall far greater than the maximum curve of rainfall for the eastern United States, as indicated by A. N. Talbot, M. Am. Soc. C. E. This curve is supposed to cover the greatest rainfall in this part of the country.

On page 184, the author cites certain storms of great intensity for 24 hours, but does not give the rate for the critical time for the watershed. Probably a rainfall of 2 or 3 hours would be much more nearly the critical period for this water-shed. Fig. 31 and Table 9 present

Mr. Gregory.

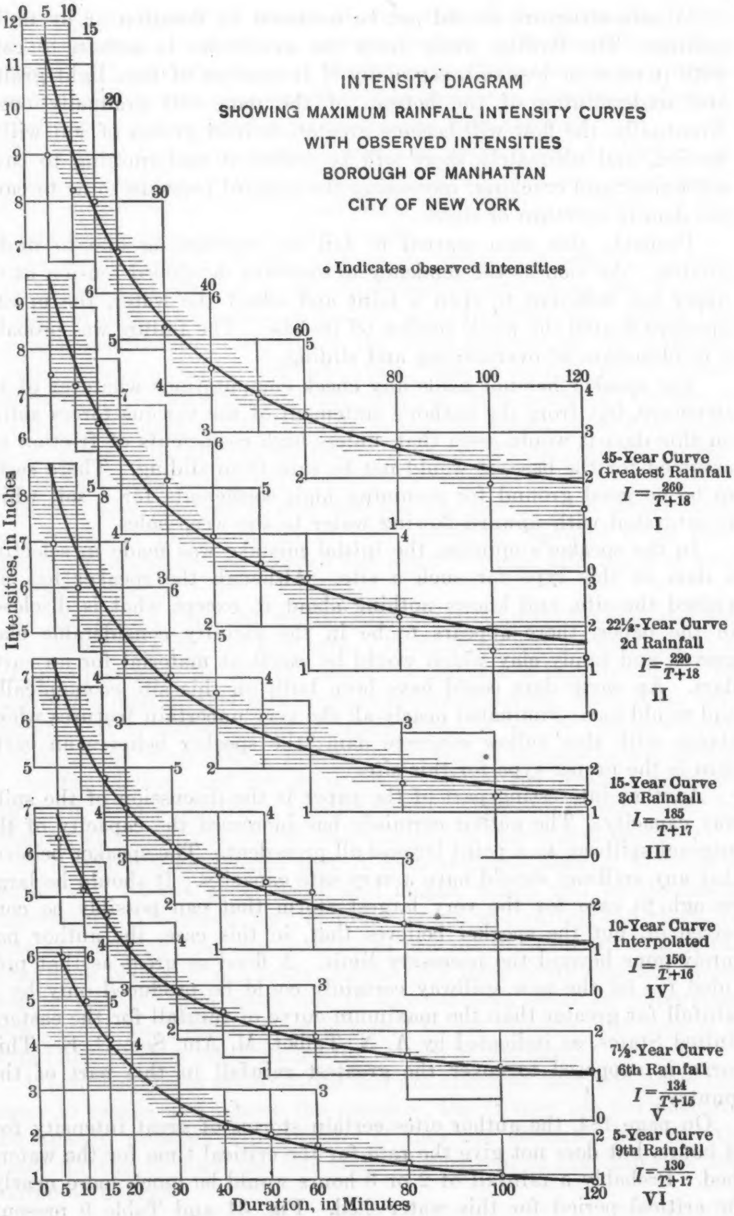


FIG. 31.

data from the automatic rain gauge in Central Park, New York, recorded continuously since 1869, and are self-explanatory. The maximum curve for Greater New York gives rates materially outside the Talbot curve, and is shown on Fig. 32. Mr. Gregory.

On the illustration of the new spillway (Plate III) is shown a joint between the buttress wall and the slab covering the bays. This joint is filled with three-ply tar-paper. Apparently, such paper was not used in the old dam, or it would not have shown the bond that it did along those joints.

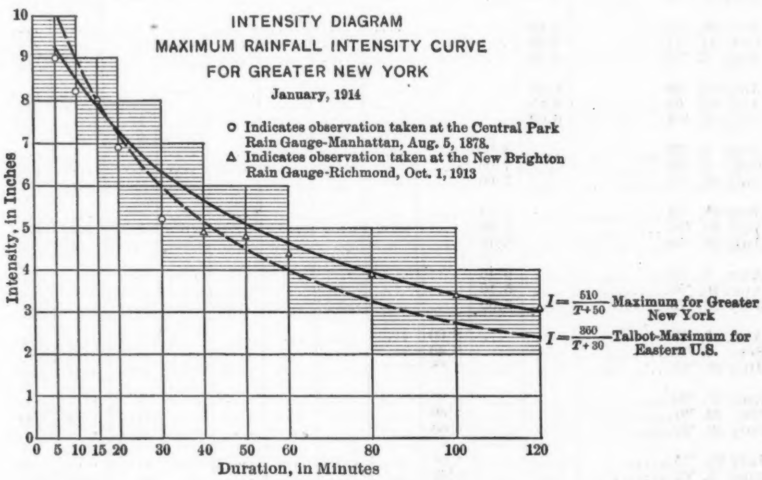


FIG. 32.

It would seem to the speaker that the placing of tar-paper in the joints of this dam, or in any dam, would be very poor practice. In the speaker's opinion, tar-paper could not be classed as a permanent material which would stand up under wet and dry, freezing and thawing, as should have been provided in a permanent structure. These joints, though they should have been made free to move, should have had some more permanent methods of stopping leakage. Even a plain, smooth joint, without any filling, would have been far better than the tar-paper. Some form of metal tongue across the joint would have been still better.

When designing the Ashokan and Kensico Dams, of the Catskill Water Supply System, the speaker had occasion to divide these gravity masonry dams into sections for the purpose of taking up the contraction due to temperature changes. In the Ashokan Dam the joint was formed with concrete blocks, as shown on Fig. 33, and the leakage through the opening of the joint was prevented by constructing a 6-in.

Mr. Gregory

TABLE 9.—MAXIMUM RATES OF RAINFALL, IN INCHES PER HOUR, AS RECORDED BY RAIN GAUGE IN CENTRAL PARK, NEW YORK CITY.

| Date. | Duration in Minutes. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5 | 10 | 15 | 20 | 30 | 40 | 50 | 60 | 80 | 100 | 120 |
| Aug. 5, '78 | 9.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 3, '92 | 7.44 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 31, '10 | 6.98 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| May 30, '12 | 6.60 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Oct. 11, '71 | 6.48 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 6, '95 | 6.36 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 12, '80 | 6.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 20, '93 | 6.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 28, '02 | 6.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '78 | .... | 8.20 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | 7.20 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 31, '10 | .... | 6.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 28, '13 | .... | 5.70 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| May 30, '12 | .... | 5.28 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 28, '02 | .... | 5.16 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '02 | .... | 5.16 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 20, '93 | .... | 5.10 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| July 26, '75 | .... | 5.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '78 | .... | .... | 8.00 | .... | .... | .... | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | 6.40 | .... | .... | .... | .... | .... | .... | .... | .... |
| July 28, '13 | .... | .... | 4.88 | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 20, '93 | .... | .... | 4.80 | .... | .... | .... | .... | .... | .... | .... | .... |
| May 22, '81 | .... | .... | 4.68 | .... | .... | .... | .... | .... | .... | .... | .... |
| July 28, '02 | .... | .... | 4.60 | .... | .... | .... | .... | .... | .... | .... | .... |
| July 26, '75 | .... | .... | 4.56 | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '02 | .... | .... | 4.52 | .... | .... | .... | .... | .... | .... | .... | .... |
| May 30, '12 | .... | .... | 4.12 | .... | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '78 | .... | .... | .... | 6.93 | .... | .... | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | 6.00 | .... | .... | .... | .... | .... | .... | .... |
| Aug. 20, '93 | .... | .... | .... | 4.35 | .... | .... | .... | .... | .... | .... | .... |
| July 28, '02 | .... | .... | .... | 4.20 | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '02 | .... | .... | .... | 4.05 | .... | .... | .... | .... | .... | .... | .... |
| July 28, '13 | .... | .... | .... | 3.93 | .... | .... | .... | .... | .... | .... | .... |
| July 26, '75 | .... | .... | .... | 3.81 | .... | .... | .... | .... | .... | .... | .... |
| May 22, '08 | .... | .... | .... | 3.57 | .... | .... | .... | .... | .... | .... | .... |
| Oct. 1, '13 | .... | .... | .... | 3.45 | .... | .... | .... | .... | .... | .... | .... |
| Aug. 5, '78 | .... | .... | .... | .... | 5.22 | .... | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | .... | 5.12 | .... | .... | .... | .... | .... | .... |
| July 28, '02 | .... | .... | .... | .... | 4.08 | .... | .... | .... | .... | .... | .... |
| Aug. 20, '93 | .... | .... | .... | .... | 3.30 | .... | .... | .... | .... | .... | .... |
| July 28, '13 | .... | .... | .... | .... | 3.16 | .... | .... | .... | .... | .... | .... |
| Aug. 5, '02 | .... | .... | .... | .... | 2.90 | .... | .... | .... | .... | .... | .... |
| Aug. 19, '04 | .... | .... | .... | .... | 2.76 | .... | .... | .... | .... | .... | .... |
| July 26, '75 | .... | .... | .... | .... | 2.54 | .... | .... | .... | .... | .... | .... |
| May 22, '81 | .... | .... | .... | .... | 2.42 | .... | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | .... | .... | 4.42 | .... | .... | .... | .... | .... |
| Aug. 5, '78 | .... | .... | .... | .... | .... | 3.93 | .... | .... | .... | .... | .... |
| July 28, '02 | .... | .... | .... | .... | .... | 3.69 | .... | .... | .... | .... | .... |

TABLE 9.—(*Continued.*)

Mr. Gregory.

| Date. | Duration in Minutes. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5 | 10 | 15 | 20 | 30 | 40 | 50 | 60 | 80 | 100 | 120 |
| July 28, '13 | .... | .... | .... | .... | .... | 2.70 | .... | .... | .... | .... | .... |
| Oct. 1, '13 | .... | .... | .... | .... | .... | 2.59 | .... | .... | .... | .... | .... |
| Aug. 20, '93 | .... | .... | .... | .... | .... | 2.49 | .... | .... | .... | .... | .... |
| Aug. 19, '04 | .... | .... | .... | .... | .... | 2.40 | .... | .... | .... | .... | .... |
| Aug. 5, '02 | .... | .... | .... | .... | .... | 2.35 | .... | .... | .... | .... | .... |
| Sep. 23, '82 | .... | .... | .... | .... | .... | 2.18 | .... | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | .... | .... | .... | 3.84 | .... | .... | .... | .... |
| July 28, '02 | .... | .... | .... | .... | .... | .... | 3.32 | .... | .... | .... | .... |
| Aug. 5, '78 | .... | .... | .... | .... | .... | .... | 3.17 | .... | .... | .... | .... |
| July 28, '13 | .... | .... | .... | .... | .... | .... | 2.88 | .... | .... | .... | .... |
| Oct. 1, '13 | .... | .... | .... | .... | .... | .... | 2.14 | .... | .... | .... | .... |
| July 6, '96 | .... | .... | .... | .... | .... | .... | 2.04 | .... | .... | .... | .... |
| Aug. 23, '97 | .... | .... | .... | .... | .... | .... | 2.04 | .... | .... | .... | .... |
| Aug. 2, '93 | .... | .... | .... | .... | .... | .... | 2.00 | .... | .... | .... | .... |
| Aug. 19, '04 | .... | .... | .... | .... | .... | .... | 1.99 | .... | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | .... | .... | .... | .... | 3.40 | .... | .... | .... |
| July 28, '02 | .... | .... | .... | .... | .... | .... | .... | 3.02 | .... | .... | .... |
| July 28, '13 | .... | .... | .... | .... | .... | .... | .... | 2.68 | .... | .... | .... |
| Aug. 5, '78 | .... | .... | .... | .... | .... | .... | .... | 2.66 | .... | .... | .... |
| Oct. 1, '13 | .... | .... | .... | .... | .... | .... | .... | 1.90 | .... | .... | .... |
| July 6, '96 | .... | .... | .... | .... | .... | .... | .... | 1.85 | .... | .... | .... |
| Sep. 23, '82 | .... | .... | .... | .... | .... | .... | .... | 1.74 | .... | .... | .... |
| Aug. 23, '97 | .... | .... | .... | .... | .... | .... | .... | 1.74 | .... | .... | .... |
| Aug. 19, '04 | .... | .... | .... | .... | .... | .... | .... | 1.72 | .... | .... | .... |
| Sep. 4, '13 | .... | .... | .... | .... | .... | .... | .... | .... | 2.70 | .... | .... |
| July 28, '13 | .... | .... | .... | .... | .... | .... | .... | .... | 2.20 | .... | .... |
| Oct. 1, '13 | .... | .... | .... | .... | .... | .... | .... | .... | 1.65 | .... | .... |
| Sep. 23, '82 | .... | .... | .... | .... | .... | .... | .... | .... | 1.62 | .... | .... |
| Aug. 19, '04 | .... | .... | .... | .... | .... | .... | .... | .... | 1.47 | .... | .... |
| July 6, '96 | .... | .... | .... | .... | .... | .... | .... | .... | 1.46 | .... | .... |
| May 21, '83 | .... | .... | .... | .... | .... | .... | .... | .... | 1.44 | .... | .... |
| July 5, '01 | .... | .... | .... | .... | .... | .... | .... | .... | 1.36 | .... | .... |
| June 29, '03 | .... | .... | .... | .... | .... | .... | .... | .... | 1.36 | .... | .... |
| Oct. 1, '13 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.88 | .... |
| July 5, '02 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.44 | .... |
| Sep. 23, '82 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.36 | .... |
| Oct. 9, '03 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.29 | .... |
| Aug. 19, '04 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.23 | .... |
| Oct. 4, '77 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.20 | .... |
| Sep. 4, '78 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.20 | .... |
| June 29, '03 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.20 | .... |
| Oct. 23, '12 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.13 | .... |
| July 5, '01 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.32 |
| Sep. 23, '82 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.31 |
| Oct. 9, '03 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.21 |
| Sep. 4, '78 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.10 |
| Oct. 4, '77 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.08 |
| Aug. 19, '04 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.07 |
| Aug. 4, '88 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1.02 |
| Aug. 23, '97 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 0.96 |
| Aug. 23, '12 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 0.96 |

Mr. Gregory.

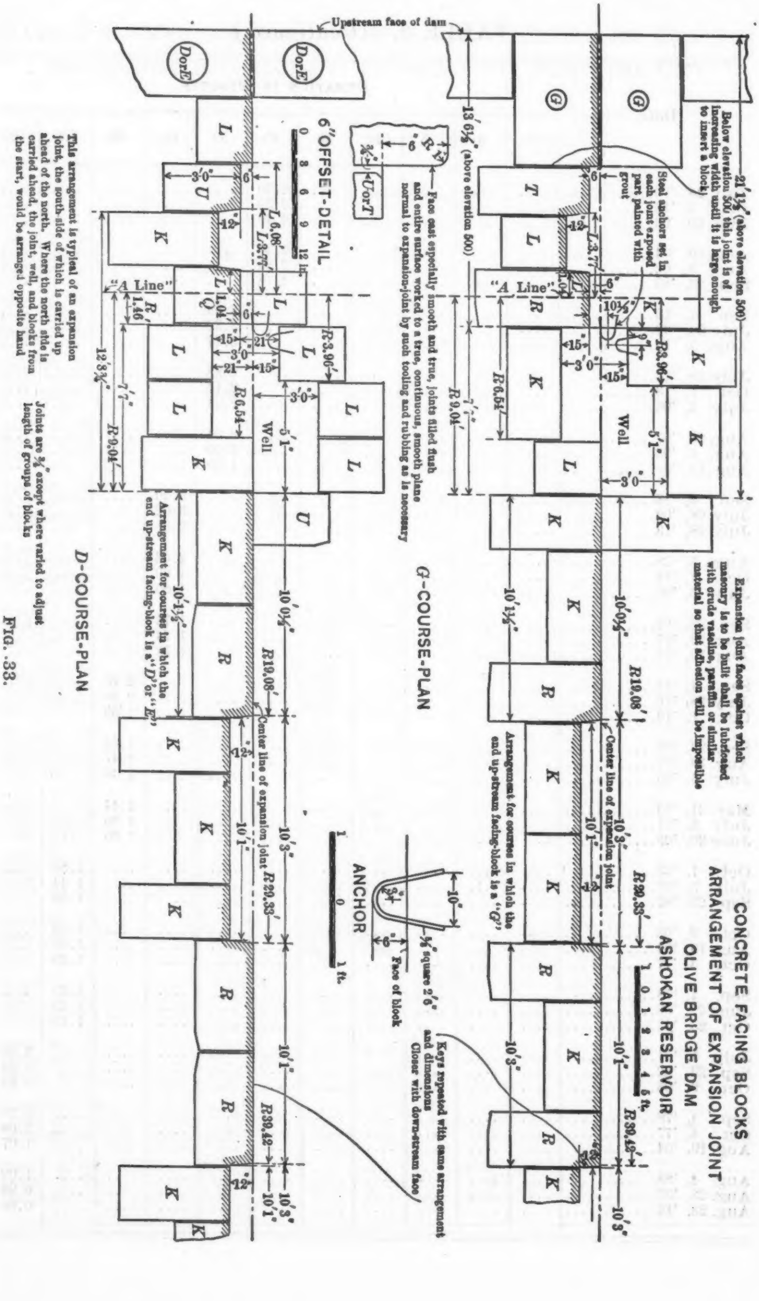


FIG. 33.

Mr. Gregory.

offset, with a face very accurately finished in a plane parallel to the longitudinal axis of the dam, so that when the joint opened the concrete surfaces would slide on each other without opening.

Just down stream from this offset, as shown, is the drainage well to collect and carry to a lower gallery whatever leakage might come through the joint. Experience has shown the leakage through the fifteen joints of this dam, aggregating about 1 500 lin. ft. of exposed joint, to have been about 450 000 gal. per day as a maximum, and to have been reduced to about 25 000 gal. per day on November 2d, 1916.

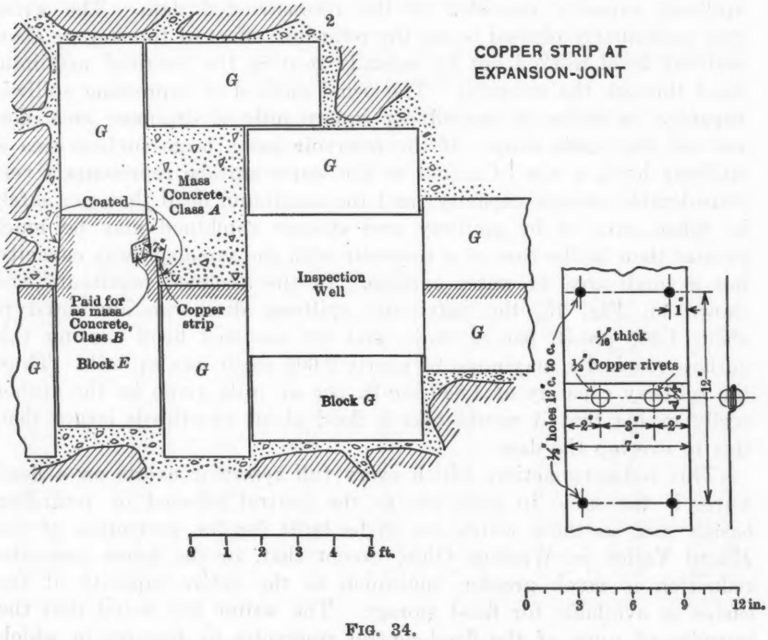


FIG. 34.

In the Kensico Dam, similar joints were provided with a copper strip arranged diagonally across the sliding joint, so that, when the joint opens, the copper strip (as shown on Fig. 34) will crumple rather than tend to pull out. These structures have been very effective in all joints but two where there was considerable leakage at first, due to a poor batch of concrete near the bottom of the well in one case, and a contraction crack below the bottom of the well and copper strips in two cases which admitted more water than the entire twenty-two joints in the dam. After these cracks were grouted the leakage was reduced to from 13 000 to 18 000 gal. per day for the 1 900 lin. ft. of exposed joint.

Mr. Grant.

KENNETH C. GRANT,* M. AM. SOC. C. E. (by letter).†—This paper is very interesting to the writer, who made a careful examination of the dam on January 16th, 1914, and agrees with the author's conclusions as to its failure. The writer is impressed by the thoroughness of the studies and designs for the repair of the dam, and cannot but feel that it is unfortunate that the same care was not taken in the original design and construction, when equal safety could have been obtained at much less cost.

All readers of this paper must have been impressed by the large spillway capacity provided in the reconstructed dam. The writer was particularly pleased to see the retarding effect of the storage above spillway level worked out by actually routing the assumed maximum flood through the reservoir. The usual method of expressing spillway capacity, in terms of run-off per square mile of drainage area, does not tell the whole story. If the reservoir has a large surface area at spillway level, a rise of a foot in the water surface represents a very considerable storage capacity; and the maximum flood that can safely be taken care of by spillway and storage combined may be much greater than in the case of a reservoir with the same spillway capacity, but a small area of water surface. In the assumed maximum case shown on Fig. 15, the maximum spillway discharge amounted to about 1 600 sec-ft. per sq. mile, and the assumed flood causing this outflow reached a maximum of nearly 2 600 sec-ft. per sq. mile. Thus, the spillway capacity of 1 840 sec-ft. per sq. mile given by the author really means that it would take a flood about two-thirds larger than this to overtop the dam.

This reducing action, which every full reservoir exerts on a flood wave, is the same in principle as the control effected by retarding basins such as those which are to be built for the protection of the Miami Valley in Western Ohio, except that, in the latter case, the reduction is much greater, inasmuch as the entire capacity of the basins is available for flood storage. The writer has noted that the capacity of some of the flood-control reservoirs in Europe, in which the conduits can be closed by gates, is correctly considered to be the total capacity up to the elevation at which the spillway discharge reaches the maximum outflow that can be safely delivered to the channel below.

It may be of interest to explain briefly the method found to be the simplest for routing floods through the Miami Valley retarding basins, by applying it to the flood wave assumed by the author in Fig. 12. In the method here to be described, the time interval is fixed, and the rate of outflow is obtained by trial. The rate of inflow is fixed by the adopted time interval, and the trial rate of storage and

* Dayton, Ohio.

† Received by the Secretary, April 23d, 1917.

Mr. Grant.

elevation of reservoir surface are fixed as soon as the rate of outflow is assumed. As compared with the method of fixing the elevation of reservoir surface and finding the time interval by trial, this method has the advantage of enabling one to pick the breaks in the inflow curve, especially the peak, and the points where inflow equals outflow.

Fig. 35 shows a capacity curve of the Stony River Reservoir above spillway level, derived from Fig. 10; also a curve showing spillway discharges plotted against reservoir capacities, derived from Fig. 11. Fig. 36 shows the inflow and outflow curves, and Table 10 illustrates the method of recording the routing operation.

TABLE 10.—CANE CREEK FLOOD OF MAY, 1901, ROUTED THROUGH STONY RIVER RESERVOIR.

(Assumed Stony River discharges derived from Cane Creek discharges by direct ratio of drainage areas. No flash-boards on spillways. Reservoir full at beginning of flood.)

| Elevation of reservoir surface. | Rate of inflow, in second-feet. | Rate of outflow, in second-feet. | Rate of storage, in second-feet. | Mean rate of storage, in second-feet. | TIME. | | STORAGE. | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Increment, in hours. | Total, in hours. | Increment, in millions of cubic feet. | Total, in millions of cubic feet. |
| 136.20 | 100 | 100 | 0 | 0 | 0 | 0 | 0 | 208.40 |
| 136.24 | 500 | 150 | 350 | 175 | 1.0 | 1.0 | 0.63 | 209.03 |
| 136.38 | 1 300 | 270 | 1 030 | 690 | 1.0 | 2.0 | 2.48 | 211.51 |
| 136.55 | 2 000 | 420 | 1 580 | 1 305 | 0.6 | 2.6 | 2.82 | 214.33 |
| 137.04 | 4 850 | 1 080 | 3 770 | 2 675 | 0.9 | 3.5 | 8.68 | 223.01 |
| 137.70 | 5 100 | 2 250 | 2 850 | 3 310 | 1.0 | 4.5 | 11.91 | 234.92 |
| 137.98 | 6 000 | 2 900 | 3 100 | 2 975 | 0.5 | 5.0 | 5.35 | 240.27 |
| 138.40 | 8 000 | 3 820 | 4 180 | 3 640 | 0.6 | 5.6 | 7.86 | 248.13 |
| 138.98 | 11 800 | 5 250 | 6 550 | 5 365 | 0.6 | 6.2 | 11.60 | 259.73 |
| 139.77 | 15 800 | 7 550 | 8 250 | 7 400 | 0.6 | 6.8 | 16.00 | 275.73 |
| 140.05 | 14 000 | 8 480 | 5 520 | 6 885 | 0.25 | 7.05 | 6.20 | 281.93 |
| 140.24 | 9 170 | 9 170 | 0 | 2 760 | 0.39 | 7.44 | 3.88 | 285.81 |

Conditions of starting similar to those given by the author, were assumed, namely, that, at 7 A. M., the outflow is equal to the inflow, or 100 sec-ft. From Fig. 35, this corresponds to a capacity of 208 400 000 cu. ft., or an elevation of 136.2.

Take a time interval of 1 hour. At this time the rate of inflow is 500 sec-ft. By trial, a rate of outflow at this time can quickly be found such that the storage increment added to the storage already existing gives a total storage corresponding to the assumed outflow. Thus, by assuming a rate of outflow of 150 sec-ft. at the end of 1 hour, the rate of storage is 350 sec-ft., and the mean rate of storage is 175 sec-ft. Multiplying this by 3 600 gives the storage increment, in cubic feet, or 630 000 cu. ft. Adding this to 208 400 000, the storage

Mr. Grant. at Elevation 136.2, gives 209 030 000 cu. ft., the total storage. Entering the spillway capacity curve on Fig. 35 with this reservoir capacity gives a discharge of 150 sec-ft., which checks the original assumption as to outflow. The elevation of reservoir surface, 136.24, is found by enter-

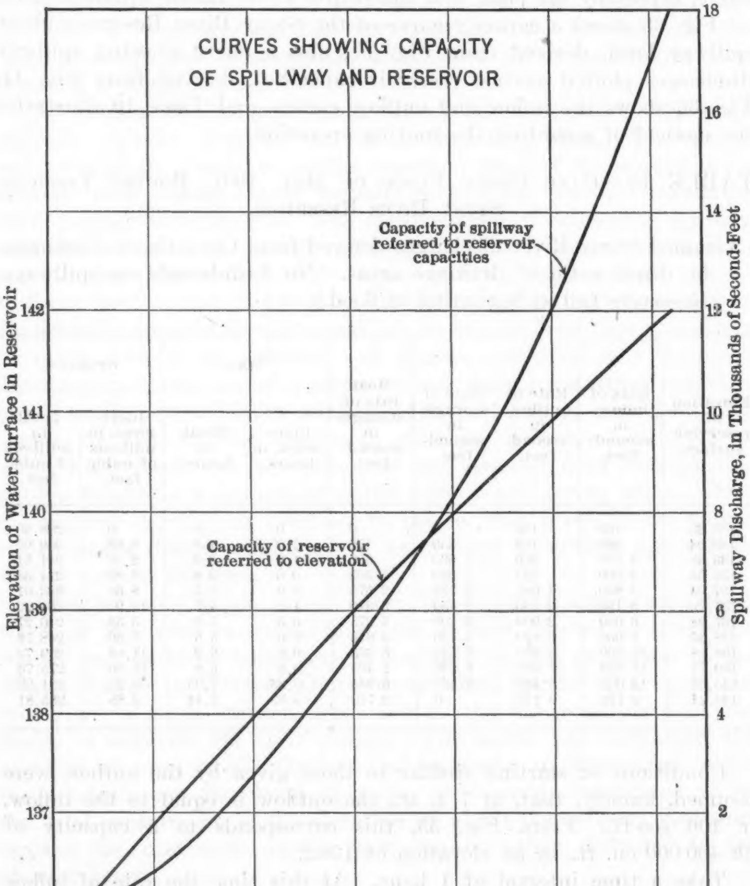


FIG. 35.

ing the reservoir capacity curve on Fig. 35 with the total capacity, 209 030 000 cu. ft.

Continuing this operation, the maximum outflow is reached, at which time the outflow is equal to the inflow, and the rate of storage

is zero. From this point until the water surface lowers to spillway level, the outflow is greater than the inflow, and the storage increments are negative. Mr. Grant.

CANE CREEK FLOOD OF MAY, 1901,
ROUTED THROUGH STONY RIVER RESERVOIR.
SHOWING REDUCTION DUE TO STORAGE ABOVE SPILLWAY.

Assumed Stony River discharge derived from Cane Creek discharges by direct ratio of drainage areas. No flash-boards on spillways. Reservoir full at beginning of flood.

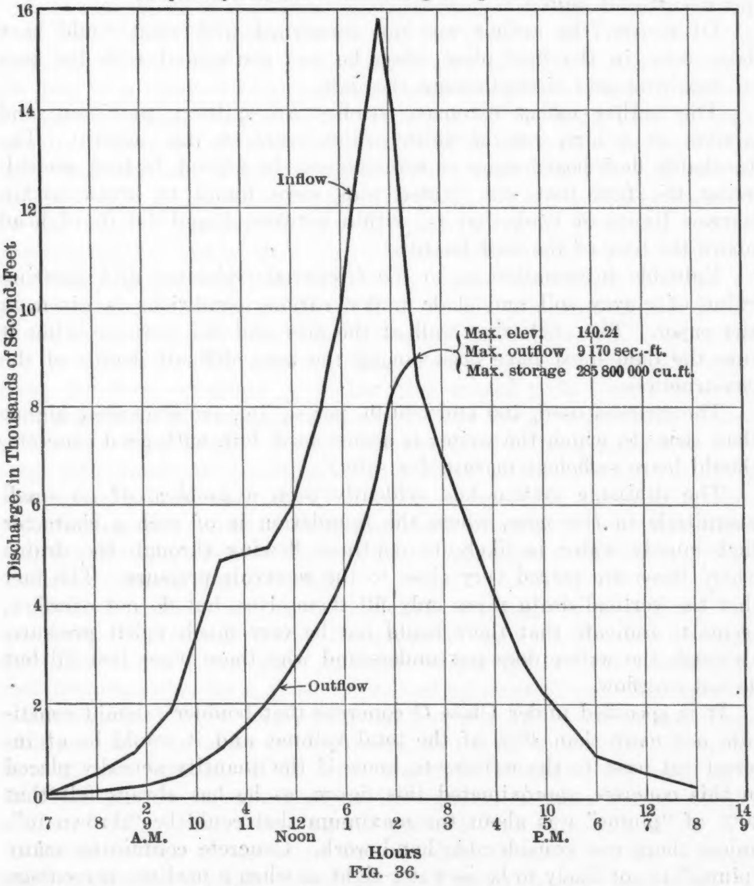


FIG. 36.

This same method can be used, of course, where the reservoir is kept entirely empty for flood control, the spillway discharge curve being replaced by a curve in which the discharge of the outlet conduits is plotted against the corresponding reservoir capacities.

Mr. Jorgensen.

L. R. JORGENSEN,* M. AM. SOC. C. E. (by letter).†—This is a very detailed analysis of the execution of a difficult piece of dam work—more difficult than the construction of the original structure.

To the writer it seems that an earth-fill dam with a concrete core-wall would have been the logical type in such a place, as there probably was enough clayey soil in the neighborhood to cover the bottom in an up-stream direction from the core-wall, although it is stated in the paper that there was not enough suitable material to build a plain earth-fill dam.

Of course, the author was not concerned with what could have been done in the first place when he was confronted with the task of repairing and strengthening the dam.

The author makes extensive studies for spillway provision, and arrives at a high run-off figure which seems to be justified. The breakable flash-board support arrangement is a good feature, considering the fact that the treated pins were found to break within narrow limits of load, that is, within between 4 and 4.6 ft. of head above the base of the flash-boards.

Valuable information as to the frictional resistance and shearing value of clayey soil and shale under various conditions is given in the paper. The anchoring wall at the heel and the work of tying it into the dam must have been among the most difficult details of the construction.

The stresses used, 700 and 500 lb. per sq. in., are somewhat higher than those to which the writer is accustomed, but, with good concrete, should leave sufficient margin for safety.

The drainage system has evidently been a problem of no small magnitude in this case, where the foundation is of such a character that muddy water is likely to continue flowing through the drains where these are placed very close to the reservoir pressure. The fact that the vertical drain pipes only fill themselves, but do not overflow, seems to indicate that there could not be very much uplift pressure, although the writer does not understand why these pipes just fill but do not overflow.

It is specified under Class *C* concrete that boulders should constitute not more than 40% of the total volume, and it would be of interest—at least to the writer—to know if the quantity actually placed in this concrete approximated this figure, as he has always felt that 20% of "plums" was about the maximum that could be "thrown in", unless there was considerable hand work. Concrete containing many "plums" is not likely to be as water-tight as when a medium percentage is used, say, 20%, which is not so small a percentage at that.

* San Francisco, Cal.

† Received by the Secretary, April 23d, 1917.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—The speaker has been greatly interested in this description of the repair and reconstruction of the Stony River Dam. The conditions were very unfavorable, and the author is to be complimented on the very thorough manner in which the work was done, giving the dam large factors of safety against all possible kinds of failure which might occur.

In order to correct some erroneous statements which have been made—some of them in print—the speaker must state his connection with the inception of this dam. He was engaged by the West Virginia Pulp and Paper Company, in the fall of 1911, to visit the site of a proposed storage reservoir in the mountains of West Virginia, which site had been selected by an engineer employed by the Paper Company for this purpose. Owing to the steepness of the river beds in that region, it was very difficult to find a suitable site for a storage reservoir, and the one finally chosen, after considerable investigation, appeared to be about the only one available. The speaker was requested by the Paper Company to give his opinion about this site, and to recommend the type of dam which he thought should be adopted.

In accordance with this request, he made one visit to the site of the reservoir, late in the fall of 1911, in the company of Mr. R. P. Bloss, the engineer of the Paper Company. At that time, only a few test pits had been excavated. They showed that the material overlying the rock consisted of yellow clay mixed with very fine sand, and underlain by compact blue clay. The rock surface was found to be at a considerable depth. The surface of the ground was covered with numerous boulders, and in most of the test pits boulders were found at a certain depth below the surface. An alarming feature was the fact that veins of some fine black material which looked like coal appeared in some of the test pits. Analyses, made later, proved that this material was not coal.

Both Mr. Bloss and the speaker were much impressed by the possibility that leakage under the proposed dam might occur through the black veins in the clay, unless a proper cut-off was provided to rock bottom, either by a masonry wall or by sheet-piling. The speaker requested Mr. Bloss to dig additional test pits and to make soundings with an auger, in order to determine, as nearly as possible, the position of rock bottom. This work was done subsequently, and, early in 1912, Mr. Bloss submitted to the speaker a cross-section of the valley of Stony River, showing the probable line of rock bottom.

Based on this cross-section and the data obtained by the test pits and borings, the speaker reported to the Paper Company as follows:

1.—That the site selected for the reservoir would be satisfactory, if the valley were closed by a well-constructed dam.

* New York City.

Mr. Wegmann.

2.—That a masonry dam would cost more than $500 000, owing to the great depth to rock, which is about 45 ft. in the center of the valley.

3.—That an earth dam was out of the question, as no earth or gravel, for mixing with the clay at the site of the dam, could be found within a reasonable distance from the proposed reservoir.

4.—That the only type of dam which could be built at the proposed site, within a reasonable sum—say, from $150 000 to $200 000—was a hollow dam of reinforced concrete.

After receiving the speaker's report, the Paper Company decided to construct a hollow dam of reinforced concrete, and requested the speaker to prepare a contract and specifications for this work. Three different companies which had had experience in this kind of construction were invited to submit plans and bids for the construction of a hollow dam of reinforced concrete, based on the cross-section of the valley prepared by Mr. Bloss. On this plan Mr. Bloss, in consultation with the speaker, had marked the least depth to which the foundation of the cut-off wall would probably be excavated, and the contract provided that if the foundation should go deeper, the additional work involved should be paid for as an extra. At both ends of the dam, where it was thought that the cut-off wall would probably not go down to rock, sheet-piling was shown.

Four different plans* for constructing the dam of reinforced concrete were received, with lump-sum bids, ranging from $143 000 to about $200 000, for building the dam to the depth shown on the cross-section of the valley.

Only one among the bidders, the Ambursen Hydraulic Construction Company, of Boston, Mass., had had much experience in the construction of dams of the proposed type. This Company had built, at that time, more than seventy dams of this kind, ranging in height up to 150 ft. All these dams were standing, and although one of them—that at Pittsfield, Mass.,† the cut-off wall of which had not been carried deep enough in a foundation of gravel—had been undermined at the center of the valley, the dam had merely sagged, but had not been ruptured. As the hole made under this dam by undermining had been 20 ft. deep, 53 ft. wide, and about 50 ft. long, both up stream and down stream, the structure had certainly shown remarkable strength. It had been jacked up, a deeper cut-off wall had been provided, and no further trouble had been experienced.

In view of the wide experience of the Ambursen Company in constructing hollow dams of reinforced concrete, the speaker strongly

* *Engineering News*, September 5th, 1912.

† *Engineering News*, April 1st, 1909.

Mr. Wegmann.

advised the Paper Company: (1) to engage the Ambursen Company to design the proposed dam; and (2), in case this Company should not get the contract for building the dam, to employ one of its experienced engineers or superintendents to be constantly on the ground while the dam was being built, in order to insure that the plans of the Ambursen Company would be properly carried out.

These recommendations were adopted by the Paper Company, and the speaker's connection with the Stony River Dam then terminated. The speaker had never seen the plans finally adopted, until after the partial failure of the dam in January, 1914.

It appears that the westerly half of the dam was constructed by the Webber Construction Company, the lowest bidder, and the easterly half was built by the Ambursen Hydraulic Construction Company. According to the plans shown to the speaker by the Paper Company, after the partial failure of the dam, and according to various published accounts, the cut-off wall was only 5 ft. deep below the floor of the dam from the west end to Buttress 15, although the depth of water at the latter point was more than 25 ft. This part of the core-wall was founded on what appeared to be hard-pan, a tough clay which was very hard to pick but became soft after being under water. For the remaining length, the cut-off wall was carried down to what was thought to be bed-rock. As the usual rule is to make the depth of the cut-off wall at least half the depth of the water, the inconsistency of the manner in which the dam was constructed was apparent, and it is difficult to understand how the engineers in charge of the work could have been satisfied with such a shallow cut-off wall, especially as sheet-piling had been omitted at the ends of the dam, although they knew that there were porous seams in the clay formation.

After the partial failure occurred, the Ambursen Company issued a bulletin about "The Facts as to the Blow-out under the Stony River Dam at Dobbin, West Virginia." In this pamphlet the Company stated that the plans submitted to them by the speaker "showed sheet-piling under each edge as a cut-off", but that its Mr. Ambursen, after visiting the site of the dam, "expressed disbelief as to the possibility of driving sheet-piling, and recommended a cut-off trench to be carried down into sound material." The pamphlet continued as follows: "The test pits were at that time* examined jointly by the owner's engineer, their consulting engineer, Mr. Wegmann, and Mr. Ambursen."

The speaker cannot understand how this statement got into the pamphlet, as it is absolutely incorrect. He never met Mr. Ambursen and Mr. Bloss at the site of the dam, and, at the time mentioned, had no connection with the work.

* April, 1912.

Mr. Wegmann.

A few days after the dam had been ruptured, Mr. A. G. Hillberg, representing the *Engineering Record,* visited the site. According to the published account of his observations,* the core-wall had only been carried down 5 ft. below the flooring of the dam, at the point where the break occurred, although the depth of the water in the reservoir at this point was more than 25 ft. Mr. Hillberg found, at the point of rupture, a "pervious seam of coal and sand with some clay as a binder, about 8 in. below the level at which the cut-off wall had been stopped." The seam was from ¼ to 6 in. thick, and about 4 ft. wide. This discovery gave a clear proof of the cause of the failure of the dam, and as Mr. Scheidenhelm, also, has stated in his report to the Public Service Commission of West Virginia that "failure was caused by the undermining of the over-burden or soil under the up-stream cut-off wall", the speaker thinks that this question might be considered as definitely settled.

Mr. Scheidenhelm has not only repaired the breach in the dam, but has made a number of important changes in its design and construction which the speaker will discuss.

*Increasing the Spillway.*—As originally built, the spillway was only 150 ft. long, and 3 ft. deep below the crest of the main dam. In case of a severe freshet, the whole dam could have acted safely as a spillway, if a suitable apron had been constructed on the down-stream side.

Assuming the probable maximum freshet at 1 386 sec-ft. per sq. mile—the figure given by the author for Cane Creek—the bulkhead section of the dam, as originally built, would have had to pass a sheet of water about 2.5 ft. deep. In all probability, the dam would have been able to pass this water, but, as no apron had been provided for the bulkhead section, the latter would have been gradually undermined.

In reconstructing the dam, Mr. Scheidenhelm increased the spillway capacity to about 1 840 sec-ft., and, although this may seem unusually large, it made the dam very safe.

*Increasing the Storage Capacity of the Reservoir.*—By placing flash-boards on the two spillways provided in the reconstructed dam, Mr. Scheidenhelm raised the water level 3.5 ft., and thus increased the reliable storage capacity of the reservoir by 25 per cent.

Of course, this increased the stresses in the original dam, and reduced its factor of safety against sliding; but part of the reinforcement put in the reconstructed dam should be charged to raising the water level 3.5 ft., and not to weakness in the dam, as originally built.

*Resistance to Sliding.*—The author made some experiments on the frictional resistance of clay moving on clay, shale on shale, and concrete on shale. In all these experiments, the coefficient of friction appears to have been determined for materials in motion. The force

* *Engineering Record,* January 24th, 1914.

required to start the motion is known to be much greater than that needed to keep a body moving. As sufficient data on these points are not yet available, the author acted wisely in adopting conservative figures. Mr. Wegmann.

The original dam had withstood successfully for about 65 days the pressure due to a full reservoir, before failure occurred on January 15th, 1915, and there was no indication that the dam had not sufficient stability against sliding. As the weepers in the floor of the dam had, doubtless, been closed by ice during this period, and as the reservoir had not, at that time, been made water-tight by silting up, the dam, in all probability, had been subjected to upward pressure. Mr. Scheidenhelm thought that a greater upward pressure might occur in the spring, when the ground was full of water, but, in the speaker's opinion, this is doubtful, as probably the weepers would be open at that season.

*Resistance to Overturning.*—One of the advantages of a hollow dam of reinforced concrete, having its deck on an angle of 45°, is its stability against overturning. As the water rose in the reservoir and finally over-topped the crest of the dam, the line of pressure was drawn up stream, so as to intersect the base of the dam near its center. The speaker thinks the original dam would not have failed by overturning. The provisions made during the reconstruction to increase the stability against sliding, at the same time, gave the dam a still larger factor of safety against overturning.

*Extension of the Cut-Off Wall.*—Mr. Scheidenhelm has extended the shallow cut-off wall near the west end of the dam to rock. His explorations by test pits and drill-holes showed that, in some places, where the cut-off was thought to have been founded on rock, it was really built on boulders, and permitted leakage under its base. In such cases, Mr. Scheidenhelm carried the cut-off deeper, and, in other places, he made, by a small V-shaped trench filled with concrete, a water-tight seal between the up-stream side of the cut-off wall and the bed-rock. The width of this wall—only 2½ ft.—did not seem to be sufficient for the maximum pressure it had to sustain, and, therefore, it is not surprising that Mr. Scheidenhelm found that in some places water leaked through the cut-off wall.

*Miscellaneous Construction.*—Mr. Scheidenhelm has strengthened the original footings of the buttresses, remedying faulty conditions in places by pressure grouting. He has also provided a proper drainage system for taking care of the leakage through the foundation soil, and has housed in the higher bulkhead portions of the original dam by curtain-walls and roofs, in order to prevent serious freezing in the drainage system, etc.

In conclusion, the speaker compliments the author on the very thorough manner in which he has repaired and improved the dam, and on the detailed account of this work which he has given in his valuable paper.

Mr. Church.

IRVING P. CHURCH,* ASSOC. AM. SOC. C. E. (by letter).†—In connection with Fig. 12, of his valuable and exhaustive paper, the author refers to a tentative process by which such a curve as *a-b-g*, or "Discharge over Spillways", is determined; that is, a curve showing the rate of discharge, $Q$, over the spillway as a function of the time; so that with $Q$ known as a function of $H$, the depth on the spillway, it becomes possible to compute $H$ for any epoch; this being a case where the rate of influx into the reservoir, or "flood discharge", is given as a function of time, in such a curve as *a-c-d*, or graph of flood discharge, in Fig. 12.

As a matter of this kind is very rarely treated in books on hydraulics, it may be of interest to consider the strict mathematical nature of such a problem, that is, where efflux takes place through an orifice or over a spillway from a very wide vessel or reservoir, simultaneously with an influx into the reservoir; and where it is desired to determine the value of the head, $H$ (head on orifice or over crest of spillway), as a function of the time, $t$.

At any instant of time, suppose the surface of the water in the reservoir to be rising, and assume the following notation:

$A =$ the area of that surface, at any instant;
$F =$ area of orifice, if one is used;
$\mu =$ coefficient of discharge of orifice;

and, if a spillway is used, let

$b =$ length of crest; and
$c =$ coefficient of discharge.

Also, let

$Q =$ rate of efflux, in cubic feet per second, at any instant;
and $Q' =$ rate of influx, in cubic feet per second, at any instant.

During any short interval of time, $d\,t$ (time-increment), $H$ increases by $d\,H$, and the gain of the volume of water in the reservoir must be equal to the excess of influx over efflux during this time; that is,

$$Q' \,.\, d\,t - Q \,.\, d\,t = A \,.\, d\,H \qquad (1)$$

(If the surface were sinking, we should have $- A \,.\, d\,H$ instead of $+ A \,.\, d\,H$.)

*Case I.*—Let $Q'$ be constant and the reservoir have vertical sides, with efflux taking place through an orifice; then $A$ is constant. Let $H_0$ be the initial value of $H$, and let the constant, $Q'$, be written in the form, $Q' = \mu\, F \sqrt{2\,g} \,.\, K^{\frac{1}{2}}$, where $K$ is an ideal constant head, easily computed. Equation (1) now becomes:

$$d\,t = \frac{A}{\mu\, F \sqrt{2\,g}} \,.\, \frac{d\,H}{K^{\frac{1}{2}} - H^{\frac{1}{2}}} \qquad (2)$$

* Ithaca, N. Y.

† Received by the Secretary, May 1st, 1917.

This is readily integrable by a temporary change of variable, $K^{\frac{1}{2}} - H^{\frac{1}{2}}$ being denoted by $Z$; so that $d\,H = -\,2\,H^{\frac{1}{2}}\,.\,d\,Z, = 2\,(Z - K^{\frac{1}{2}})\,.\,d\,Z.$ Mr. Church.

Now, $H = H_0$ for $t =$ zero and, finally, after integration, the form is obtained,

$$t = \frac{2\,A}{\mu\,F\,\sqrt{2\,g}}\left[\sqrt{H_0} - \sqrt{H} + \sqrt{K}\,.\,\log._e\left(\frac{\sqrt{K} - \sqrt{H_0}}{\sqrt{K} - \sqrt{H}}\right)\right]..(3)$$

giving the time, $t$, as a function of $H$. If $t$ is given and $H$ sought, resort must be had to solution by trial. (By "$\log._e$" is meant Naperian logarithm.)

*Case II.*—Let the reservoir have a rectangular spillway on which initially (that is, for $t =$ zero), the head is $H_0$ though at any later instant it is $H$, the rate of efflux being then $Q = c\,.\,b\,\sqrt{2\,g}\,.\,H^{\frac{3}{2}}$. Let the rate of influx, $Q'$, be constant, and let an expression be written for it in the form, $Q' = c\,.\,b\,\sqrt{2\,g}\,.\,K^{\frac{3}{2}}$, where $K$ is an ideal constant head. The sides of the reservoir are vertical; hence $A$ is a constant.

Assuming that $Q'$ is greater than the initial value of $Q$, we have from Equation (1):

$$d\,t = \frac{A}{c\,.\,b\,\sqrt{2\,g}}\,.\,\frac{d\,H}{K^{\frac{3}{2}} - H^{\frac{3}{2}}}\,\ldots\ldots\ldots\ldots\ldots(4)$$

The integration* of this equation is quite roundabout, but finally† leads to the result

$$t = \frac{A\,K}{3\,Q'}\left[\log._e\left(\frac{[\sqrt{K} - \sqrt{H_0}]^2\,(H + \sqrt{H\,K} + K)}{[\sqrt{K} - \sqrt{H}]^2\,(H_0 + \sqrt{H_0\,K} + K)}\right)\right.$$

$$\left. -\,\sqrt{12}\,\tan.^{-1}\left(\frac{(\sqrt{H} - \sqrt{H_0})\,\sqrt{12\,K}}{3\,K + (2\,\sqrt{H_0} + \sqrt{K})\,(2\,\sqrt{H} + \sqrt{K})}\right)\right].\,(5)$$

in which, as before, $\log._e$ signifies "Naperian logarithm of"; and $\tan.^{-1}$ denotes "anti-tangent of", or "arc whose tangent is"; for example, $\tan.^{-1}\,0.488 = 0.454$ (since $\tan.\,26° = 0.488$, and $26°$ expressed in arc ("radians") is 0.454).

*Case III.*—Rate of influx, $Q'$, not constant, but proportional to the time. Efflux over a spillway. If $Q'$ is a linear function of $t$, that is, is proportional to $t$, when $t$ is reckoned from a special origin, we have a case that is suggested by the graph of the Cane Creek flood of the author's Fig. 12; since this curve, *a-c-d*, etc., may be considered to be made up of a number of consecutive straight lines. Among the more

* Detail will be found on p. 200 of Frizell's "Water Power", First Edition.

† On pp. 362 and 430 of *Engineering News*, November and December, 1901, will be found interesting matter relative to this, by the Messrs. Gould.

Mr. Church. prominent of these straight lines is the portion, $c$-$d$; and this portion has been treated by the writer in the attempt to discover the law connecting the variables, $H$ and $t$, holding good for any instant of time between about 9.30 and 10.30 A. M.

If a straight-edge be applied to the diagram in Fig. 12, it will be found that the prolongation of the straight line, $d$-$c$, cuts the time-axis very closely at 9 A. M.; and that the point, $d$, corresponds to a value of $Q' = 5\,000$ cu. ft. per sec. at 10.30 A. M., that is, 5 400 sec. along the time-axis from 9 A. M. Hence, if we reckon $t$ from 9 A. M., we have the proportion, $Q' : 5\,000 :: t : 5\,400$, or $Q' = 0.926t$ (for the foot and second as units).

Since the elevation of the surface of the water at this stage of the flow is less than 140 ft., only two of the three spillways will be in action, namely, the "old" and the "new"; and these have the same crest elevation of 136.0 ft. and hence are equivalent to a single spillway for which the rate of discharge, or "capacity" (see Fig. 11), is

$$Q = 3.65\,(150 + 133)\,.\,H^{\frac{3}{2}};$$

that is,

$$Q = 1\,033\,.\,H^{\frac{3}{2}} \text{ cu. ft. per sec. (with } H \text{ in feet).}$$

Again, in Fig. 10, it is to be noted that the curve of reservoir capacity is nearly straight between Elevations 134.0 and 138.0, the gain of volume in that interval of 4 ft. being 500 000 000 gal. or a fairly constant rise of 125 000 000 gal. per ft. Division by 7.58, and by 1 ft., gives 16 710 000 sq. ft.* as a fair estimate of the area, $A$, in the region of Elevation 136.0. These expressions and values having been inserted in Equation (1), there is obtained, after reduction and division (for foot-second units),

$$10\,000\,000\,\frac{d\,H}{d\,t} = 0.554\,t - 618.4\,H^{\frac{3}{2}} \qquad \text{.............(6)}$$

This differential equation not admitting of a strict mathematical solution,† resort was had to plotting a curve with $H$ as ordinate and $t$ as abscissa ($t$ is reckoned from 9 A. M. and $H$ is the head on the spillway), a special method being used for the purpose, giving a very close approximation.

It is a property of a circular arc that the tangent lines drawn at its extremities intersect at a point equi-distant from the points of tangency, and the method in question consists virtually in drawing, "tandem", a number of very flat circular arcs, the radius of each being the average radius of curvature for the small extent of curve involved.

* From the curve "area in acres" of Fig. 10, we might also have taken $A = 386$ acres $\times$ 43 560 = 16 820 000 sq. ft.

† Information on this point was kindly furnished by James McMahon, Professor of Mathematics, Cornell University.

The arcs themselves are not drawn, being sufficiently defined by the extremities and their tangents. Mr. Church.

For each of the five values of $H$: 0.6, 0.7, 0.8, 0.9, and 1.0 ft., the value of the derivative, $\frac{d\,H}{d\,t}$, was computed from Equation (6) for each of some six or eight values of $t$, chosen so that, by previous inspection, the true value of $t$ for the assumed $H$ would probably lie within the range taken; and these thirty or forty values were tabulated.

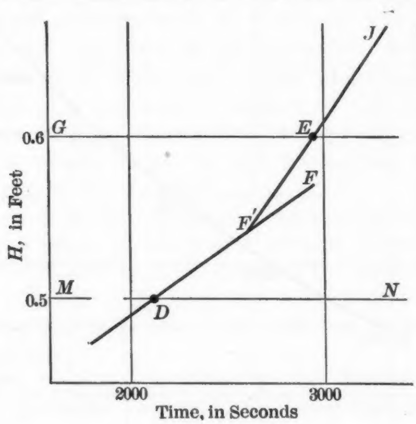


FIG. 37.

In Fig. 37, for the starting point, $D$ (corresponding as regards time to points, $b$ and $c$ of the author's Fig. 12), we have $H = 0.50$ ft., and $t$ will be taken as 2 034 sec. (that is, from 9 A. M.; see Fig. 12). Also, the value of the "slope" of the tangent line at $D$, namely, $\frac{d\,H}{d\,t}$, is found from Equation (6) to be $\frac{0.0910}{1\,000}$. A straight line, $D\,F$, is then drawn through $D$ at such an angle that a portion having 1 000 sec. as horizontal projection has 0.0910 ft. as vertical projection. Use was made of co-ordinate paper with twenty divisions to the inch, and the scales adopted were 1 000 sec. to 5 in. horizontally, and 0.10 ft. to 2.5 in. vertically.

A point, $E$, is then found by trial on the horizontal line, $GE$, that is, for $H = 0.60$ ft., such that for the value of $t$ for this point, $E$, the straight line drawn through $E$ and having a slope, $\frac{d\,H}{d\,t}$, obtained from Equation (6) for these values of $H$ and $t$, intersects the previous tangent line, $DF$, in a point, $F'$, making $\overline{F'D}$ equal to $\overline{F'E}$. We have thus determined very closely a new point, $E$, of our curve and the

Mr. Church.

tangent line, $F'EJ$, at this point. A similar procedure involving the tangent, $E\ J$, and the horizontal line for $H = 0.7$ ft., gives another point on the curve and also its tangent line; and so on.

The five points thus obtained in addition to the starting point, $D$, are plotted in Fig. 38 and give the curve there shown. The figures annexed to the points give the abscissas (time), in seconds.

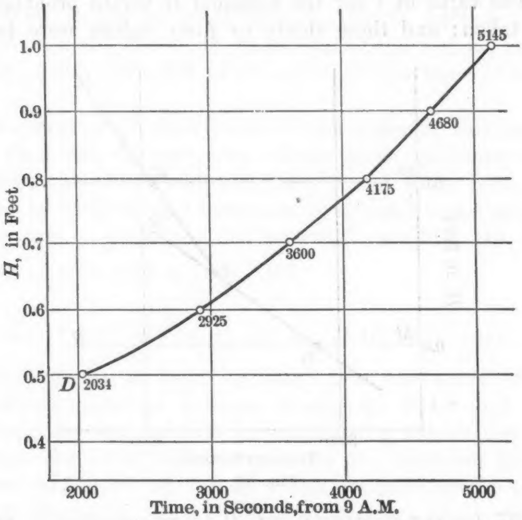


FIG. 38.

Investigations involving a change of origin and the plotting of the logarithms of the new co-ordinates, brought out the fact that the relation

$$H = 0.30 + \left(\frac{0.025}{10\,000\,000}\right)(2\,000 + t)^{2.19} \ldots\ldots\ldots (7)$$

is practically the equation to this curve, within the limits shown; giving results not more than 1% in error when $H$ is solved for. The $t$ of this equation is reckoned from 9 A. M., as before.

For the point, $d$, in the author's Fig. 12, the value of $t$ is 5 400 sec. and this value, in Equation (7), gives $H = 1.04$ ft. For this point the author has 137.0 ft. as elevation of the surface; that is, $H =$ just 1.00 ft.; and, under the circumstances, this may be considered a very fair agreement of the two methods.

Mr. O'Shaughnessy.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E. (by letter).†—The author is to be commended for the thorough manner in which the

* San Francisco, Cal.

† Received by the Secretary, May 1st, 1917.

reconstruction of this dam was undertaken. The complete way in which the details of the construction and the reasons therefor have been described are also worthy of commendation. Mr. O'Shaughnessy.

To a Western engineer, the extraordinary flood flow per square mile, of the region which drains into the dam, appears to be alarming. It would seem that the new spillway capacity, more than six times the original size, should surely be ample to underwrite the structure against flood damage in the future. It would also seem that this hollow, reinforced concrete type of dam was well adapted for this particular site, and that its original failure was due, not to the type of structure, but, in a large way, to lack of adequate care with the foundation portion of the work. Neglect of this nature is likely to cripple any dam, and one of this type perhaps less than any other, as a fracture of a couple of panels will release the stored waters and safeguard the rest of the structure.

One of the cardinal principles to be observed in any dam foundation is to remove the uplift pressure and make a perfect contact between the water-tight skin or face of the dam and the solid reliable foundation material. The results of tests on the friction resistance of clay soil and the analysis of the sliding factors furnish data of original labor, which are creditable.

The method of pressure grouting the footings is a matter of interest, and apparently this work was very well done in the reconstruction.

The observation that "great care is necessary even in foundation work, where it is too often assumed that any kind of work will suffice", should be impressed on the minds of all young engineers as the most important of all the elements that need attention. No matter how perfect the superstructure, if the foundation is inadequate, final failure or consequent trouble will result. If the leakage on May 25th, and November 1st, 1915, was actually measured, the facts would be of interest.

The reason for the construction of a curtain-wall and roof to avert the freezing of the top of the weep-holes and deep drains, is a matter of much interest to engineers working in a country which is practically frostless.

The author's comments on cracks in the original structure from lack of proper contraction joints is illuminating. The writer's experience has been that where large masses of concrete were exposed to rapid temperature changes, there should be joints at least 40 ft. apart, and, in ditch lining, he has used joints as close as 12 ft. to advantage.

With reference to asphalt in expansion joints, there is a tendency of this material to flow in hot weather. Such a condition was studied

Mr. O'Shaughnessy.

in the reconstruction of the Twin Peaks Reservoir, in 1912, and by adding a percentage ($17\frac{1}{2}\%$ by weight) of diatomaceous, kieselguhr, or infusorial (different names for the same substance, which is almost pure silica), to the asphalt, this tendency to flow was removed, and the asphalt still retained its plastic condition.

In stating the cost per cubic yard, the details of the cement and iron costs are missing, and would be interesting if added by the author.

The writer has seen no paper, for some time, which has been so candid in every manner in describing all the engineering features, so that brother engineers can profit by the experiments and experience gained in this structure.

# MEMOIRS OF DECEASED MEMBERS.

**NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.**

## CHARLES CONRAD SCHNEIDER, Past-President, Am. Soc. C. E.*

DIED JANUARY 8TH, 1916.

Charles Conrad Schneider, the son of Julius and Emilie Schneider, was born in Apolda, Saxony, on April 24th, 1843. He attended school in his native city, and then served an apprenticeship in a machine shop. After this he attended the Royal School of Technology in Chemnitz, Saxony, from which he was graduated in 1864. He afterward devoted a few years to active professional practice as a Mechanical Engineer.

In 1867, Mr. Schneider came to the United States and, for three years, was employed as a Draftsman with the Rogers Locomotive Works, at Paterson, N. J.

He was quick to perceive the great field for development which lay in the branch of the profession which he subsequently adopted, and his name will be prominently associated for all time with those members of the profession who have done so much in the past 40 or 50 years to study, develop, and perfect the science and art of Structural Engineering.

His first work in this line was with the Michigan Bridge and Construction Company, of Detroit, Mich., with which Company he accepted a position as Assistant Engineer in 1871. In 1873, he took charge of the Engineer's Office of the Erie Railroad Company in New York City, under the late Octave Chanute, Past-President, Am. Soc. C. E., Chief Engineer. In this position, one of Mr. Schneider's duties was to check the strain sheets and plans submitted by bridge companies. Up to this time this had not been the general practice, the railroads depending mainly on the bridge companies for the correctness of the designs. He also organized a thoroughly trained inspection force. At this time experienced inspectors could hardly be obtained, and the work was generally accepted without inspection. Mr. Schneider selected a number of young men from the office and shops, who showed practical ability, and gave them instructions which enabled them to inspect the work in the shops, both as to quality and progress.

Bridge work up to this time had usually been let on a competitive lump-sum basis. Mr. Schneider soon found that this method was

* Memoir prepared by the following Committee: Paul L. Wolfel, Richard Khuen, Jr., and O. E. Hovey, Members, Am. Soc. C. E.

unsatisfactory, and the Railroad Company's officials decided to make their own plans; and it was Mr. Schneider's duty to prepare them. The bridge requirements for 1874 were designed in this way, and the work was let on a pound-price basis, and this, in all probability, was the first instance where bridge work was let by the pound, a procedure which Mr. Schneider always favored.

In 1876 and the early part of 1877, he was engaged by the Board of Engineers (sometimes called the Steinway Commission), appointed by the Long Island Bridge Company, to prepare and pass on plans for a railroad and highway bridge across the East River, to connect Long Island with New York City. It was the intention to locate this bridge between 76th and 77th Streets, this site having been selected as the narrowest point in the East River. The plan required a 734-ft. span across the west channel and a 618-ft. span across the east channel. The project, however, was not carried out at this time, on account of the financial condition of the Company.

From May, 1877, to July, 1878, Mr. Schneider was associated with the Delaware Bridge Company, of New York, as a Designer. Charles Macdonald, Past-President, Am. Soc. C. E., was President of this Company. During this period the Delaware Bridge Company made an arrangement with the Edge Moor Iron Company, of Wilmington, Del., to manufacture the ironwork of the bridges built by the former company, and Mr. Schneider was stationed at Edge Moor where he had charge of the design and construction of various bridges, including the Rockville Bridge over the Susquehanna River, near Harrisburg, Pa., on the Pennsylvania Railroad, the Cohoes Bridge, on the Delaware and Hudson Railroad, and several smaller ones.

On August 1st, 1878, he opened his own office in New York City, specializing in the design of bridge and structural work. One of the first pieces of work handled by him was a number of Howe truss spans for the Canadian Pacific Railway, the most important of which was for the Stony Creek Viaduct. This work was done by Mr. Schneider for the Contractor, Mr. Andrew Onderdonk, and this bridge carried traffic for the Canadian Pacific Railway until 1893, when it was replaced by a large arch span. Mr. Schneider's connection with the Canadian Pacific Railway Company dates from this time. For this Company he acted as Consulting Engineer, in numerous cases, almost to the end of his life.

One of the most important structures for this railroad was the 527-ft. cantilever bridge over the Fraser River, built by Mr. Schneider in 1887. This was one of the first cantilever bridges built in America, and it carried traffic until 1910, when it was taken down to make room for a structure of greater capacity; but it was re-erected across a chasm known as Niagara Ravine, on the line of the Esquimault and Nanaimo Railway (operated by the Canadian Pacific Railway),

near Victoria, B. C. Both these structures were considerably ahead of their time, and have not been improved on by the most recent practice, except in a very few features.

In more recent years, Mr. Schneider passed on the design of the Lethbridge Viaduct, for the Canadian Pacific Railway, and also quite a number of smaller structures, including some Scherzer and Strauss bascule bridges. He was also called in consultation by this Railway Company for the reconstruction of the St. Lawrence River Bridge, near Montreal, Que., where the rebuilding of the superstructure and the remodeling of the piers for a wider and heavier bridge involved numerous difficult and interesting problems.

Numerous other structures were handled by Mr. Schneider in his New York office, the most important of which was the cantilever bridge over the Niagara River for the Grand Trunk Railroad. He wrote a highly interesting paper* about this structure for the Society, for which he was awarded the Rowland Prize in 1886.

He also designed all the interior steel framework and the anchorage for the Statue of Liberty in New York Harbor during this time.

When, in 1886, competitive designs were requested for a crossing over the Harlem River, by the Commission appointed by the City of New York, Mr. Schneider was awarded first prize for his plans. His friends have always regretted that his very beautiful and correctly conceived design had not been built in place of the structure which is now known as the Washington Bridge.

From 1879 to 1883, Mr. Schneider was associated with the late George S. Morison, Past-President, Am. Soc. C. E., on a number of important structures, such as the Plattsmouth, Bismarck, and Blair Bridges across the Missouri River, and the Snake River Bridge, at Ainsworth, Wash.

In 1886, Mr. Schneider was a member of a board of consulting engineers appointed by the New York District Railway Company. His associates on this board were: William P. Trowbridge, Charles C. Martin, Julius W. Adams, John T. Fanning, Alfred P. Boller, Gen. Quincy A. Gilmore, Henry Morton, and Charles F. Chandler. This Company proposed to build an underground railway under Broadway, with a junction at 14th Street, one line passing up Madison Avenue and the other continuing north under Broadway.

In May, 1886, Mr. Schneider entered into an agreement with the A. and P. Roberts Company, of Philadelphia, Pa., owners of the Pencoyd Iron Works, to establish a Bridge and Construction Department in connection with its works, and, subsequently, was appointed Chief Engineer. Under his direction this Bridge and Construction Department developed into the largest and most progressive establish-

* "The Cantilever Bridge at Niagara Falls", *Transactions*, Am. Soc. C. E., Vol. XIV, p. 499.

ment of its kind in the United States, and gained an international reputation.

Some of the most important structures built by the Pencoyd Iron Works under his supervision are the Delaware River Bridge, for the Pennsylvania Railroad, near Philadelphia; the Niagara Falls and Clifton Arch Bridge, over the Niagara River, near the Falls; the Pennsylvania Railroad Company's old trainshed, in Jersey City; the Chesapeake and Ohio Bridge down the James River, in Richmond; and innumerable smaller structures in the United States, Mexico, and Japan.

In 1893, the Long Island Railroad Company took an interest in the project for a bridge over the East River at Blackwell's Island, previously referred to, with a view of establishing a terminal in New York City. Work was actually commenced, and, in 1894, competitive designs for the bridge were invited. The bridge was to accommodate four railroad tracks, with approaches and a terminal station west of Second Avenue, New York City. The plans adopted by the Long Island and New York Bridge Company were the designs made by Mr. Schneider, for the Pencoyd Iron Works, to which Company the contract for the entire steel superstructure of the bridge and approaches was awarded in March, 1895. Considerable work had been done on this bridge on piers and foundations, complete detailed drawings of the superstructure were made, and a portion of the material was rolled, when, on account of the death of Austin Corbin, President of the Long Island Railroad, the Company again became disorganized.

The American Bridge Company was formed in 1900, and, on May 21st, Mr. Schneider was elected Vice-President in charge of Engineering. He held this office until May 16th, 1901, when it was abolished and, on the same day, he was elected a Director and Vice-President of the American Bridge Company of New York, also in charge of Engineering. These offices he held until April 20th, 1903, when he became Consulting Engineer of the Company, which position he held during the remainder of his life. His associates in the American Bridge Company held him in the highest esteem, and felt that his influence for good in the Company was very potent, particularly in the early days of the organization. The first President of the Company expresses his estimate of him as follows:

"Mr. Schneider without question stood at the very head of his Profession and, in addition, I believe, never had an enemy in his entire career. I say this from intimate personal contact extending over a period of fifteen or twenty years. He had the confidence, not only of his fellow engineers, but of the consumer as well. His position at the head of the Engineering Department of the American Bridge Company gave to the organization a solid foundation among its competitors and the confidence of its customers. To the internal working

of the Company, he was a great advantage at the start, as naturally many conflicting interests had to be considered, and I believe the entire staff was always willing to accept his decisions without friction."

During this time, Mr. Schneider continued his connection with the Canadian Pacific Railway as Consulting Engineer, and was also appointed Consulting Engineer for the Baltimore and Ohio Railroad Company.

In 1903, he was commissioned, in conjunction with Theodore Cooper, M. Am. Soc. C. E., by the Imperial Government Railways of Japan, to prepare a large set of standard plans for bridges for the Japanese Railways. These plans are still in use, and large tonnages have been shipped from the United States to Japan in conformity with them.

After the collapse of the Quebec Bridge, he was commissioned, by the Canadian Government, to make a report on the causes of the failure, which report he finished, in 1908, in the most thorough and exhaustive manner. In 1911, he was appointed by the Canadian Government as a member of the Board of Engineers for the rebuilding of the Quebec Bridge, which position he held until his death.

Mr. Schneider frequently contributed to technical papers. Besides his article on the cantilever bridge at Niagara Falls in 1886, previously mentioned, for which he received the Rowland Prize, in 1905 he received the Norman Medal from the Society for his paper on "The Structural Design of Buildings",* and, again, in 1908 the Norman Medal for his paper on "Movable Bridges."† These, together with his "Standard Specifications for Railway and Highway Bridges" and the volume of "Standard Details" which he compiled for the American Bridge Company, known to every structural engineer, form his chief contributions to technical literature.

When Mr. Schneider wrote his first railroad specifications for the Pencoyd Iron Works, he put his impact theory into practical use for the first time. This method of calculation has been adopted by the Government of India and by a large number of American railways, and has also been incorporated in the "Manual" of the American Railway Engineering Association and in its "General Specifications for Railway Bridges."

Mr. Schneider was a member of the American Railway Engineering Association, American Society for Testing Materials, the Verein Deutscher Ingenieure, in Germany, and the Engineers' Club of New York.

Mr. Schneider was dearly beloved by his many friends on account of his sterling character and his kindly disposition. He was always willing and ready to assist brother engineers with advice, giving to them freely from his rich fund of knowledge, and large indeed is the

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 371.

† *Transactions*, Am. Soc. C. E., Vol. LX, p. 258.

number of engineers to-day in responsible positions, who owe their training and their positions to him. He was most democratic in his ways and of a lovable disposition, and gained, in the highest degree, the respect of everybody who came in contact with him. He always stood for good work, good designs, and good details, and the Engineering Profession is greatly indebted to him for the present high standard that has been obtained in bridge and structural work. His was a most useful life, well lived, an example and an inspiration to the Profession, that will remain in the memory of all who had the privilege of knowing him.

He was married on January 8th, 1880, to Catherine Clyde, daughter of John J. and Ruth H. (Luther) Winters, of New York City. It was a great blow to him when he lost his only son in early boyhood. He is survived by his widow and his daughter, Helen, the wife of John Phillips Badenhausen, M. Am. Soc. C. E.

Mr. Schneider was elected a Member of the American Society of Civil Engineers on February 6th, 1884. He served as a Director in 1887 and from 1898 to 1900, and was elected Vice-President in 1902 and President in 1905. He was Chairman of the Library Committee in 1903 and Chairman of the Committee on Concrete and Reinforced Concrete from 1904 to 1911.

## DANIEL WHEELER BOWMAN, M. Am. Soc. C. E.*

DIED MARCH 14TH, 1917.

Daniel Wheeler Bowman, the son of Quaker parents, Francis Bowman and Elizabeth (Hammond) Bowman, was born in New Bedford, Mass., on October 2d, 1844. In 1857 his parents purchased a farm at Ledyard, Cayuga County, N. Y., and moved there. Mr. Bowman spent much time on this farm, lending to his life the enthusiasm and interest which marked him throughout his later years. There he acquired and developed his great love for Nature and growing things; and this clung to him until his death.

He received his early education at Oakwood Seminary, Union Springs, N. Y., from which he was graduated in 1865. He returned and took a post-graduate course at the Seminary, on finishing which he was given charge of a Friends Academy, in Ohio. In 1868, Mr. Bowman went to Kansas on some survey work, returning the same year

* Memoir prepared by N. R. McLure, M. Am. Soc. C. E., with the assistance of Seymour P. Thomas, Esq.

to enter Cornell University where he took the Civil Engineering Course. His life at Cornell was characteristic of the man: simple, honest, and earnest. With an unusually alert mind and great application, he stood very high in all his classes, and by his native kindness and interest in his fellows and in all that pertained to the general welfare, he endeared himself to his classmates and all who knew him. He was graduated in 1872 in the first "through" class in Civil Engineering. In 1883, Cornell University conferred on him the degree of C. E.

Immediately after his graduation, Mr. Bowman was offered a position as Instrumentman on some survey work for the New York Central Railroad Company in New York State, which he accepted, and in which position he continued for two years, resigning to become associated as Engineering Secretary to the late Capt. James B. Eads, M. Am. Soc. C. E., who at that time was in charge of important jetty work at the Southwest Pass of the Mississippi River. While on this work, Mr. Bowman was directly under the late G. W. R. Bayley, M. Am. Soc. C. E., and was sent to New York City and Washington, D. C., on special missions in connection with his work. He also had charge of the construction of the dredge, *G. W. R. Bayley*, in Pittsburgh, Pa., in 1877, for use on the jetty work. At that time it was the largest dredge built, and was an engineering triumph. While on this work Mr. Bowman married Ida Portia, the daughter of Mr. Bayley. Two sons and a daughter by this marriage survive him.

After this work was finished, Mr. Bowman made some very extensive computations for a proposed bridge across the East River at Blackwell's Island, New York City, for Capt. Eads, in connection with the competition then asked for by those contemplating this structure. His ability as a Designing Engineer on this work was so marked that he was sent by Capt. Eads to Phœnixville, Pa., in connection with some work that was then being done for him by Clarke, Reeves and Company (now The Phœnix Bridge Company). At that time the late Adolphus Bonzano, M. Am. Soc. C. E., was Chief Engineer of this concern, and was so attracted by the quick perception, ability to grasp and solve difficult situations, and general engineering ability of young Bowman, that he arranged for him to remain at Phœnixville as his Assistant, in charge of estimating and designing. While in this position Mr. Bowman designed the first Kinzua Viaduct, for the New York, Lake Erie and Western Railroad Company, in McKean County, Pa., which, at that time, was one of the longest and highest railroad viaducts in the world. He designed the Albany and Greenbush Bridge across the Hudson River, at Albany, N. Y., a double-deck structure carrying railroad and roadway, with approach spans and a rim-bearing draw-span. He also designed the difficult reverse curve on the New

York Elevated Railway work, at 110th Street, Boulevard Crossing. This was built with Phœnix columns and latticed girders, and, at the time, was considered quite a complicated structure. Another notable bridge designed by Mr. Bowman at this period was that over Rondout Creek, on the West Shore Railroad.

In 1885, Mr. Bowman resigned his position as Designing Engineer of The Phœnix Bridge Company and went South, doing some bridge inspection work for the Southern Railway, and, in 1888, he was made Bridge Engineer for the Central of Georgia Railway. While in this position he had charge of the rebuilding of many of the old bridges on the line. In 1891, he was made Assistant Engineer of The Boston Bridge Works, at Boston, Mass., which position he filled until 1894, when he returned to Phœnixville, Pa., as Assistant Engineer for The Phœnix Iron Company. In 1896, he went to Chicago, Ill., with several associates, and organized the Universal Construction Company, of which he became Chief Engineer. In 1897, he again returned to Phœnixville as Assistant Engineer for The Phœnix Iron Company, under H. H. Quimby, M. Am. Soc. C. E., then Chief Engineer. On the resignation of Mr. Quimby from this Company in November, 1900, Mr. Bowman succeeded him as Chief Engineer, in charge of the estimating, computing, designing, detailing, and erection of the structural steel fabricated by The Phœnix Iron Company. He continued as Chief Engineer until February, 1914, when he was made Consulting Engineer, which position he held at the time of his death.

In 1899 he was married to M. Virginia Vanderslice, of Phœnixville, Pa. The years of Mr. Bowman's best service were devoted to structural steel, particularly as applied to bridge and building work, and there are few localities of size in the eastern section of the United States which have not, however unconsciously, felt the influence of his ability through structures erected in their midst. These were improved in construction by his ever watchful and able supervision of the design.

Mr. Bowman was broad and liberal in his views of life, a progressive in more than one sense of the word, charitable in his nature, and a friend and counsellor to those who sought his help and advice. Many a young engineer has been started on the right path through his careful and painstaking efforts in his behalf. He was a great lover of Nature, and spent many days in his later life, in the open among his flowers and fruit trees. He had an intense and loyal affection for Cornell, his Alma Mater, and seldom missed returning at Commencement time to renew his youth with the younger Alumni. Among a host of friends and acquaintances, rarely was there a man who will be so sadly missed.

Mr. Bowman was elected a Member of the American Society of Civil Engineers, on December 3d, 1912.

## ANDREW CHASE CUNNINGHAM, M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1917.

Andrew Chase Cunningham, the son of Thomas and Celeste (Chase) Cunningham, was born at Mohawk, N. Y., on February 15th, 1858.

He was appointed to the United States Naval Academy from the Twenty-first District of New York State, on June 9th, 1874, and was graduated from that institution as a Midshipman on June 10th, 1879. He served on the U. S. S. *Shenandoah* and the U. S. S. *Saratoga*, and on February 1st, 1883, he resigned from the service, meanwhile having been promoted to the rank of Ensign. Mr. Cunningham had decided by this time that a career in the Navy was not what he desired, and, having determined to become a Civil Engineer, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1885 with the degree of Civil Engineer.

From October, 1885, to April, 1886, Mr. Cunningham occupied the position of Topographer on preliminary and location surveys of the Lincoln Branch of the Missouri-Pacific Railway, and from May, 1886, to September, 1887, he served as Draftsman with the Massillon Bridge Company, Massillon, Ohio. From September, 1887, to November, 1890, he was in charge of the inspection of iron and steel in Pittsburgh, Pa., and vicinity. This work consisted in the acceptance or rejection of material for such structures as the high bridge across the Mississippi River, at St. Paul, Minn., the Ohio Connection Bridge across the river below Pittsburgh, the New York Elevated Railway, numerous large buildings in Chicago, Ill., and bridges on the Lake Shore and Michigan Southern Railway, the Pennsylvania System, the Louisville and Nashville, and other railroads.

From November, 1890, to May, 1892, Mr. Cunningham was Chief Inspector for Carnegie, Phipps and Company, of Pittsburgh, Pa., now the Carnegie Steel Company. In this position he had charge of the testing and inspection of steel materials, together with special investigation and special supervision of material for several structures, such as the Memphis Bridge, the Sixth Street Bridge across the Allegheny River at Pittsburgh, and others.

In May, 1892, Mr. Cunningham associated himself with Charles F. Stowell, M. Am. Soc. C. E., at Albany, N. Y., under the firm name of Stowell and Cunningham, the principal engineering business of the company being in connection with the design, inspection, and testing of steel bridges and steel materials. This work included materials and bridges for the New York Central and Hudson River Railroad, and the Central Vermont Railroad, as well as materials for the Cities of

* Memoir prepared by F. T. Chambers, M. Am. Soc. C. E.

Albany, N. Y., Waterbury, Conn., for the State Engineer of New York, and the U. S. Treasury Department. In 1898, while he was a member of this firm, the Spanish War broke out, and, as a former officer of the Navy, he obeyed his country's call, volunteering for such duty as he might be assigned to by the Navy Department. At that time the Navy was by no means as large as it is now, and had almost no auxiliary ships for the purpose of furnishing supplies to the fleet. Mr. Cunningham, therefore, found himself an Ensign, on May 14th, 1898, aboard the Collier *Abarenda,* a ship purchased for that use. While this ship was moored to the wharf at the New York Navy Yard, the late L. L. Buck, M. Am. Soc. C. E., visited the Navy Yard for consultation work, in connection with one of the dry docks then under repair. Some one had informed him that Mr. Cunningham was serving as an Ensign aboard the collier, and Mr. Buck expressed the determination to visit him before he left the yard, adding at the same time that he considered Mr. Cunningham one of the foremost steel experts of the country, and that it seemed to him a waste of valuable services to have placed him as an Ensign aboard a supply ship. Mr. Cunningham's nature was such that he would never have sought to be transferred from this position, but Mr. Buck felt strongly on the subject, and made it his business to inform the Navy Department of his views, the consequence being that on May 21st, 1898, Mr. Cunningham was transferred to the Bureau of Ordnance, and was immediately assigned to the Washington Navy Yard, which is the Naval gun factory. Mr. Buck and Rear-Admiral Mordecai T. Endicott, U. S. N. (*Retired*), Past-President, Am. Soc. C. E., were classmates at Rensselaer Polytechnic Institute, and it was not long before Admiral Endicott, then Chief of the Bureau of Yards and Docks, learned that an accomplished Civil Engineer was available for naval duty, and, consequently, Mr. Cunningham was transferred to the Bureau of Yards and Docks on June 27th, 1898. The Civil Engineer Corps of the Navy was then rapidly expanding and in need of officers, and Mr. Cunningham served first on a board to examine candidates for the position of Civil Engineer in the Navy; before the successful candidates were appointed, however, he himself, on September 29th, 1898, was given a permanent commission. On October 5th of that year, he reported to the Bureau of Yards and Docks, where he served until November 6th, 1901, being then detached and ordered to the Naval Station, New Orleans, La. This was a new station just being established, and Mr. Cunningham was the first Civil Engineer officer, thus being given the opportunity to lay out the engineering works from the start. On April 3d, 1903, he was detached from the New Orleans Station and ordered to the Naval Academy, where he served until June 9th, 1905, and from there he was again ordered to the Bureau of Yards and Docks. On March 17th,

1906, he was commissioned with the rank of Lieutenant in the Corps of Civil Engineers.

Mr. Cunningham continued to serve in the Bureau of Yards and Docks as Principal Assistant to Admiral Endicott until April 6th, 1907, when he was ordered to the Navy Yard, Norfolk, Va., as Senior Civil Engineer Officer, and on November 18th, 1909, he was commissioned with the rank of Lieutenant Commander. On February 20th, 1910, he was detached from the Norfolk Navy Yard, and ordered to the Navy Department, Washington, D. C., as Inspector of Public Works for the entire Navy. This duty required great tact and diplomacy, and it was on this account that Mr. Cunningham was selected for the work, which necessitated his maintaining headquarters in Washington, and visiting the various Navy Yards, keeping the Department informed as to the status of the various public works, and co-ordinating the ideas of the Yards with those of the Department.

On July 10th, 1913, Mr. Cunningham left headquarters at Washington to assume the duties of Public Works Officer of the Navy Yard, Portsmouth, N. H. Shortly afterward he had a severe nervous breakdown, and on November 17th, 1913, was ordered to sick leave, and did not return to actual duty until June 16th, 1914. He never fully recovered from this illness, although he performed lighter duties practically up to the time of his death, his principal assignments after this being at the Naval Training Station, Great Lakes, Ill., and the Naval Radio Station, Point Isabel, Tex., at both of which places he erected large steel towers for radio-telegraphy.

While on duty in the Bureau of Yards and Docks of the Navy Department, and previous to his detail as Civil Engineer Officer at New Orleans, Mr. Cunningham had supervised the construction of the 16 000-ton steel floating dock for the New Orleans Station. He became much interested in docks of this type, and, at a later date, obtained letters patent on a floating dock of his own invention. He was, indeed, of an inventive turn of mind and secured patents on several of his ideas, one of the best known in the Navy being that for a coal-tar paint.

Mr. Cunningham was affectionately known as "Andy" by his friends and associates, and was universally liked. His genial nature, combined with his diplomatic spirit, caused him to be much in demand on boards of officers for the adjustment of disputes or for changes in contracts.

While at the Naval Academy, he was the champion fencer of his time, and he maintained his interest in this sport up to the time of his severe illness. While in Washington he was a member of the Washington Fencers Club, and when he was at the Navy Yards he stirred the younger men to a revival of the fencing game. He was looked on by the entire Navy as an authority on this subject, and was also consulted by the Army at one time, in connection with the modification

of the Army saber. As a fencer, he was also interested in singlestick, and was the author of a book entitled "The Cane as a Weapon." Fencing was a considerable feature of his recreation; he was also very fond of writing, and contributed various articles to the press, among them being several on naval matters published by the Naval Institute.

Mr. Cunningham was married, at Middleville, N. Y., on June 18th, 1879, to Miss Jessie E. Thomas. He is survived by his widow and two sons: John Howard Cunningham of Grand-Mère, Canada, and George Thomas Cunningham, of Washington, D. C.

Mr. Cunningham was elected an Associate Member of the American Society of Civil Engineers on September 2d, 1891, and a Member on October 3d, 1894.

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## EDMUND HAZEN DRURY, M. Am. Soc. C. E.*

### Died January 31st, 1917.

Edmund Hazen Drury was born at St. John, New Brunswick, on July 31st, 1859. He was graduated with honor from the Royal Military College, Kingston, Ont., in June, 1881, and immediately joined the Engineering Staff of the Canadian Pacific Railway Company, continuing thereon until 1888, first as Rodman and finally as Division Engineer.

From 1889 to 1890 Mr. Drury served as Assistant Engineer on railway surveys for the Dominion Government in New Brunswick, and from 1890 to 1893, he was Division Engineer and Acting Chief Engineer on the McLeod Branch of the Calgary, Edmonton, and McLeod Railway.

From 1893 to 1906, he held the following positions: Chief Engineer and Manager of Construction for the contractors of the Quebec Central Railway, 1893-95; Division Engineer on the Canadian Northern Railway, 1895-97; Division Engineer on the Bangor and Aroostook Railway, 1897-98; Assistant Chief Engineer on the Canadian Northern Railway, 1898-1900; Assistant Chief Engineer, Cuba Company Railway, Cuba, 1900-02; Assistant Chief Engineer, Algoma Central Railway, 1902-03; Division Engineer, Crow's Nest Pass Branch, Canadian Pacific Railway Company, 1903-05; and Division Engineer, Canadian Northern Manitoba South Eastern Railway, 1905-06.

Mr. Drury then went to Mexico where he was employed as Auditing Engineer and Assistant Chief Engineer in charge for the Mexican Light and Power Company, Mexico, in Necaxa and the City of Mexico, from 1906 to 1908.

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* Memoir prepared by E. J. Walsh, Esq.

In the autumn of 1908, he was appointed, by the Department of Railways and Canals of Canada, Engineer in charge of exploration and the reconnaissance survey of alternative routes to Port Churchill and Port Nelson on Hudson Bay. His report, which was an able and comprehensive one, was concluded and handed in toward the end of 1909.

In 1910, he was appointed Chief Engineer of the Quebec and Sherbrooke Railway, which position he held until 1911, when he was engaged to prepare an estimate and report for a proposed railway from Edmonton, Alberta, to Bella Coola, on the Pacific Coast of British Columbia, *via* the Pine River Pass, Rocky Mountains, for the Edmonton, Dunvegan, and Pine Pass Railway Company.

Mr. Drury then went to South America as Chief Engineer and Manager of Construction of the Chili Longitudinal Railway, Chili, from 1911 to 1913. It was while conducting this work, through the arid nitrate district of Chili, that his health became impaired. Returning to Canada, in the autumn of 1913, he entered into a general consulting engineering partnership (in the firm of Walsh and Drury, Consulting Engineers), with an office at Ottawa, Ont.

From the latter part of 1914 to the time of his sudden death on January 31st, 1917, Major Drury had served as Acting Assistant Director General of Engineer Services, for the Department of Militia and Defence, at Ottawa.

Major Drury stood in the forefront of the profession as a Railway Engineer. He was a man of high character and unswerving integrity, and neither in private nor public affairs would he deviate from the conscientious discharge of life's duties. Kindly and courteous to a degree, he will be greatly missed by those of his confreres and others who were intimate with him.

Major Drury was a Member of the Canadian Society of Civil Engineers, and was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

[illegible]

From the latter part of 1910 to the time of his sudden death on January 31st, 1911, Major Henry had served as Acting Assistant [illegible] General of [illegible] Services for the Department of Militia and Defence, at Ottawa.

[illegible]

Major Henry was a Member of the Canadian Society of Civil Engineers, and was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

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# PROCEEDINGS

OF THE

# AMERICAN SOCIETY

OF

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

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